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Journal of the

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Proceedings of the American Society of Civil Engineers

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Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

TRANSLATIONS OF FOREIGN LITERATURE ON HYDRAULICS

Progress Report of the Task Force on List of Translations of the Committee on Hydromechanics of the Hydraulics Division

INTRODUCTION

In an attempt to bring to hydraulic engineers an up-to-date listing of trans-ions of foreign literature on hydraulics, it is the intent of the task force ᵊparing this report to issue, from time to time, progress reports containing table items. Interested readers are urged to submit discussions to (a) add this list, (b) offer suggestions for improvement, and (c) otherwise assist task force in fulfilling its aims. The present list is the first list issued ce the publication, in 1957, of ASCE Manual 35 entitled "A List of Trans-ions of Foreign Literature on Hydraulics." This list is to be considered as addendum to Manual 35 and will be included in a revision of that Manual ᵊn the time is deemed appropriate.

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Respectfully submitted,

Jan C. Van Tienhoven, Chairman
Task Force on List of Translations of the
Committee on Hydromechanics of the
Hydraulics Division

Journal of the

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OPERATION OF SPILLWAYS IN NORTHWEST PROJECTS[a]

R. B. Cochrane,[1] M. ASCE

SYNOPSIS

Experience has demonstrated that large tainter gates have many advantages ver the vertical-lift type of gates for spillways on the Columbia River multi-urpose dams, based on operation of Bonneville-, The Dalles-, McNary-, and :hief Joseph Dams. Operation experience has also shown that stilling basins f such spillways, which are subject to year around use, should be designed ith ample proportions to permit latitude in operation procedures and to mini-ize maintenance costs. Difficulties with erosion in concrete high-head con-uits in spillways point to the desirability of steel lining such conduits. Chute-ip type of spillways are being advantageously utilized in narrow valley rojects where the use is infrequent and the design capacity is not large.

INTRODUCTION

The Columbia River has its source in the Rocky Mountains in Canada, and fter flowing some 460 miles enters the United States in the northeastern orner of the State of Washington. It crosses the State in a southerly di-ection to the mouth of its largest tributary, the Snake River, near Pasco, en turns westerly, forming the boundary between the States of Oregon and ashington. At Portland, Oregon another large tributary, the Willamette iver, joins the Columbia. The Columbia River empties into the Pacific cean some 100 miles downstream from Portland. It has a total length of 210 miles and a drainage area of 259,000 square miles. Within the limits of e United States the river has a drop of about 1.7 feet per mile. At Bonneville am, the farthest downstream of the Columbia River dams, the mean flow of

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a Presented at the October 1958 ASCE Convention in New York. N. Y.

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the river is about 200,000 c.f.s., and the average annual spring flood flow from melting snows near the headwaters is approximately 585,000 c.f.s. The maximum flood of record was 1,170,000 c.f.s. in 1894; the largest flood since the construction of the Dam was 1,020,000 c.f.s. in June 1948. Low flows of 60,000 c.f.s. to 100,000 c.f.s. occur in the winter months and are often accompanied by floating ice.

Within the Columbia River Basin, the Corps of Engineers, Bureau of Reclamation and private interests have constructed many dams, a number are now under construction, and still others are in the planning stage. (See Fig. 1). The Corps of Engineers operates four of the six dams now in operation on the main stem of the Columbia, namely, Bonneville Dam, The Dalles Dam, McNary Dam and Chief Joseph Dam. These are multipurpose dams involving power, navigation, and fish passage facilities. Two of these four dams have vertical-lift type of spillway gates; the other two have tainter gates in their spillways—thus giving a good basis for comparison. On the tributary Willamette River, the Corps of Engineers has two high-head multipurpose projects in operation, namely, Detroit Dam with its reregulating Big Cliff Dam, and Lookout Point Dam with its reregulating Dexter Dam. These dams are primarily for flood control, but also have power as a secondary benefit. They both have tainter gate spillways and valve controlled conduits which are used for release of flood waters. The bulk of the operational data upon which this paper is based was obtained from the four Columbia River projects and the two Willamette River projects mentioned above. Bonneville Dam, having been in operation for the longest period of time offers the best opportunity to study spillway operations. Hydraulic model studies were made of various features of McNary-, Chief Joseph-, The Dalles-, and Lookout Point Dams, as well as the Detroit conduits, at the Bonneville Hydraulic Laboratory; model studies were made of the Detroit Spillway at the Waterways Experiment Station at Vicksburg, and Bonneville model studies were conducted at the Linton Hydraulic Laboratory.

Bonneville Dam

Bonneville Dam is located on the Columbia River approximately 40 miles east of Portland, Oregon and some 140 miles above the mouth of the river. It consists of a spillway dam, powerhouse with 10 units aggregating 518,400 KW, a navigation lock and fishways. The project was constructed in the period 1933 - 1943 at a cost of about $87,000,000. It was designed and constructed and is being operated and maintained under the supervision of the Corps of Engineers. The operating head at the dam is 26 to 62 feet. The Columbia River at Bonneville is split by an island, and the spillway is on the right side of this island. The spillway is a concrete gravity structure some 1450 feet in overall length, with an ogee spillway crest at elevation 24,* surmounted by eighteen 50-foot wide vertical lift gates. (See Fig. 2). The normal pool elevation is 72 and this elevation is maintained for all river flows up to 260,000 c.f.s. During the flood season, the pool is raised from 72.0 to 82.5 so as to maintain the rated 50-foot head on the powerhouse turbines. The spillway is designed for a free overflow discharge of 1,600,000 c.f.s. at a pool elevation of 82.5. The tailwater elevations at the spillway range from 8 at low flows to

*All elevations are in feet above mean sea level.

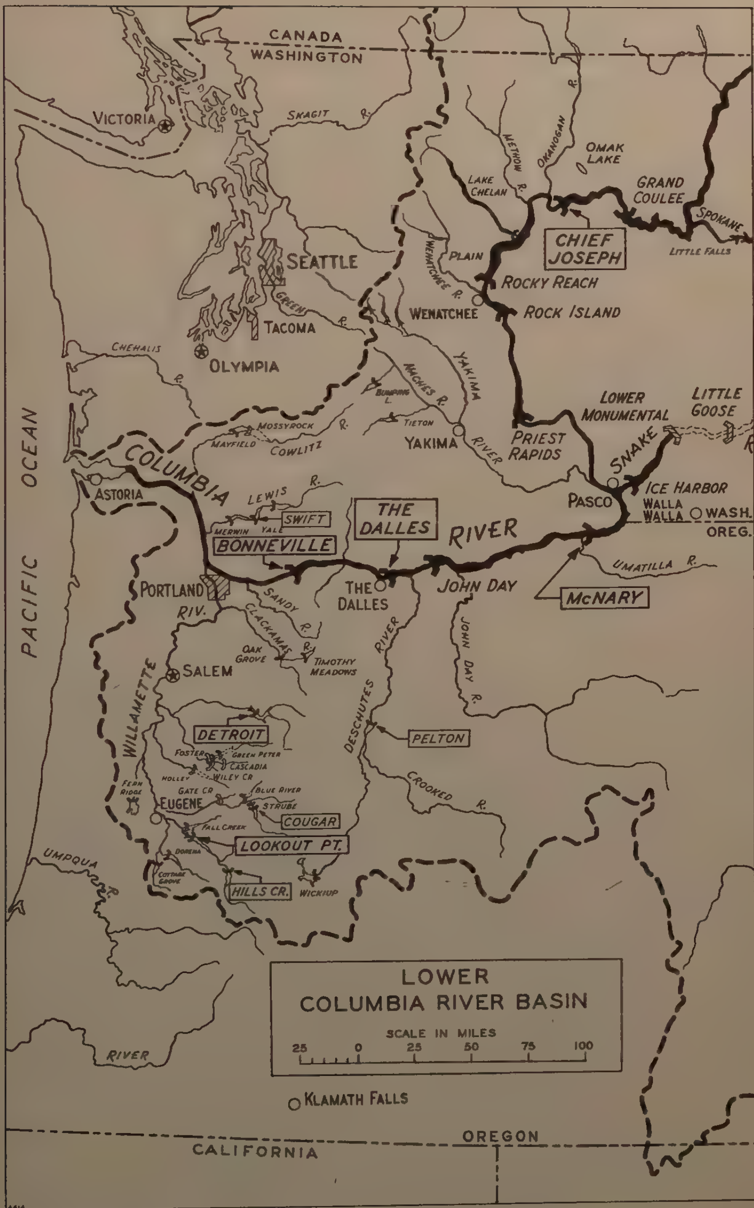
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Figure 1

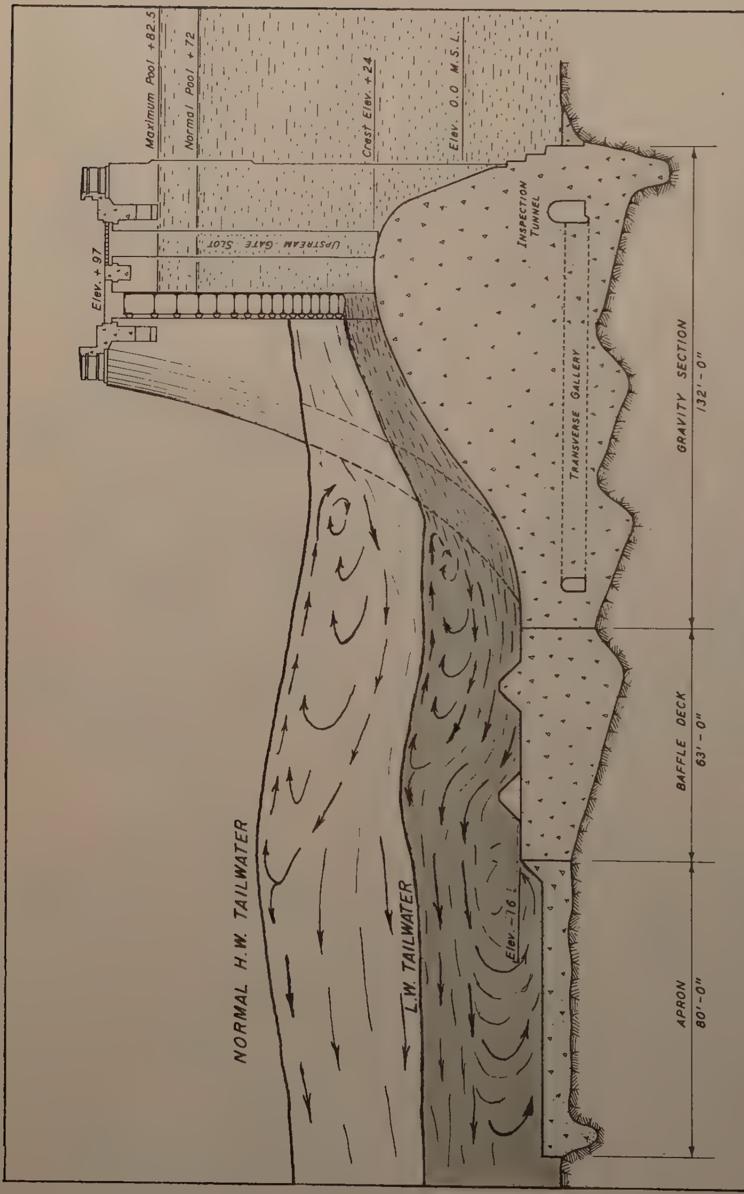


Figure 2
Bonneville Spillway Dam

about 40 at normal high water, with 56 having been attained in the 1948 flood. (See Fig. 3).

The spillway gates are constructed of riveted structural steel with roller-bearing wheels as shown in Fig. 4. They are in two sections with special lifting and latching mechanisms so that either half or the complete gate may be moved at one time. Normally the gate is in the downstream gate slot and the two gate sections are joined into a single gate unit, with flow passing beneath the bottom section of the gate. No provisions were made in the design to pass flow between the two gate sections. The gate latches or "dogs" in the piers support the gate at various gate openings by engaging "ladder" supports on the ends of the gates. These supports are at 23-inch intervals except for the first few openings. Raising and lowering of the spillway gates in the slots in the piers is by means of one of two 350-ton gantry cranes which travel along the deck of the spillway dam. These gantries are relatively slow with a gate raising and lowering speed of 2 feet per minute. On the average it takes about 25 minutes for a crane to unlatch from one gate, raise the lifting beam to deck level, move to another gate bay, lower the lifting beam, engage another gate and adjust that gate.

The spillway gates of the 3 bays at each end of the spillway are 60 feet in height, with top and bottom sections each 30 feet high. The gate can thus be closed or slightly opened and the top of the gate will still be above the maximum pool of 82.5. Usually a small gate opening is required in these end bays to counteract the eddy action along the river bank below the spillway, and create an attraction flow for the fish ladder entrances at either end of the spillway. This method of operation results in the bottoms of these gates being

Fig. 3. Bonneville Spillway in Operation—1948 Flood

submerged during high water when the tailwater exceeds 24. Larger gate openings to avoid such submergence are not desirable, since the resulting increased gate discharges would interfere with the proper functioning of the fishladders, and tend to accelerate downstream bank erosion.

The spillway gates in the 12 center bays of the 18-bay spillway dam are only 50 feet in height, with a 20-foot bottom section and a 30-foot top section. The river flow, tailwater elevations, discharge capacity of the gates and the raising of the pool are so interrelated, that these twelve gates must be raised by an amount equivalent to the rise in tailwater, with the tops of the gates just above pool level, and the bottoms of the gates at about tailwater elevation. Hence, the bottoms of these 12 center gates are seldom submerged by tailwater. Considerable operational agility is required in juggling these 12 gates to the more open positions as the pool is raised from 72 to 82.5. If the gates are not open enough, the pool rises and flows over the tops of the gates; if the gates are open too much, the pool falls below the scheduled elevation. With the latching or dogging arrangement on these gates, the minimum change in gate opening on one gate is 23 inches or one "dog" or about 3400 c.f.s. This will change the pool level some .07 feet in one hour. Rapidly changing river flows, varying powerhouse load, and desire to keep the gate openings uniform to better distribute the flow in the stilling basin create an operational problem. If the twelve 50-foot gates had been constructed at a 60-foot height like the end bays, the complications of overtopping the gates would not exist since a gate could then be closed even at a high pool if desired. At present there is no way of closing a 50-foot gate during the higher pool except to

Fig. 4. Bonneville Spillway Gate

place the 60-foot spare gate in the upstream slot which is a two hour job even with 2 cranes.

Spillway Gate Slots at Bonneville Dam

The spillway gate slots at Bonneville Dam are some 7 feet wide and 2 feet deep. The pier wall downstream from the slot is in line with the wall upstream from the slot and no setback, taper or bevel exists. The 4-inch rounded corners of the slot are armored with steel. The thrust of the gate through the wheels is taken on the bearing plate embedded in the downstream end of the slot. The "jay" rubber side seal is at the upstream edge of the gate.

Soon after the spillway was put into operation it was noticed that the concrete at the base of the piers and just downstream from the gate slot was eroded. The jet of water issuing from beneath the gate impinged on the downstream side of the gate slot and was diverted around the armored corner, but failed to adhere to the concrete pier wall downstream therefrom. This created cavitation in this area and the erosion of the concrete as shown in Fig. 5. After each high water these eroded areas were patched with various types of concrete and admixtures, but none were successful in withstanding the attack. Accordingly in 1941, steel armor plates 1/2-inch thick and 5 feet wide were installed on the pier walls and welded to the corner armor. A 2-foot width plate was also installed on the horizontal ogee surface adjacent to the wall. These armor plates were attached to the concrete with anchor bolts but the thin plate prevented high pressure grouting. As a result these plates were torn off by the water flow and had to be replaced in 1945 with one-inch steel armor plate which could be successfully anchored and grouted in place. Generally these one-inch plates have withstood the cavitation attack during the succeeding years, but small amounts of patching are still periodically required along the edge of the plate on the ogee section where it joins the concrete. Thought has been given to increasing the width of the ogee armor plate and sawing the concrete to avoid patching along the edge of the installed plate, but difficulties in matching curvature of the plate and the concrete make this procedure inadvisable. It is generally concluded that the minor extent of patching now required does not warrant too much expense in making permanent repairs.

Spillway Gate Bottom Failures

The bottom rubber seal of the Bonneville spillway gate is at the upstream 1/3 point of the gate width and the "V" shape of the bottom is rather wide with the bottom plates about 16 degrees with the horizontal. During the flood of 1943, the Bonneville pool was maintained at 82.5 for some 47 days with a peak river flow of 540,000 c.f.s. Tailwater was some 12 feet above the spillway crest elevation. During this period the 50-foot gates in the center of the spillway were open some 16 feet and hence the bottoms of these gates were not submerged. Four of the 60-foot gates in the end bays, however, were operated with small gate openings, and the bottoms of these gates were submerged by tailwater. During this time it was noted that these 60-foot gates vibrated and bounced noticeably—as much as 1/8 inch. When the gate was supported by the gantry cables, the cables became alternately slack and taut. No damage to the gate was noted at this time. During the following year vibration and bouncing of these slightly opened end gates were again discernable. Inspection after the flood receded showed that four of the gates had been damaged to some

Fig. 5. Gate-slot Damage at Bonneville Dam

extent and one gate was severely damaged as may be seen in Fig. 6. The steel fairwater or skin plate forming the bottom of the gate downstream from the rubber seal had been ripped off entirely, and most of the bottom horizontal girder (consisting of 1/2 inch web, two 6 x 6 x 13/16 inch flange angles, and the 3/4 x 11 inch cover plate) had been torn out. These gates were repaired and heavier fairwater plates and stiffeners were installed to give the gate bottoms more rigidity. During the following flood one of the repaired gates was again damaged under similar operating conditions and was repaired a second time.

In 1946 and 1947 attempts were made to aerate the bottom of the gate downstream from the seal. The later tests even included the generous perforation of the fairwater plate and the installation of fifteen 8-inch to 12-inch vent pipes extending from the pocket below the lower girder up to atmosphere. The fairwater plate downstream from the seal was even removed entirely. During these experiments water would spurt up one vent pipe to a height of 30 feet, and some 3 feet away, air would be whistling down the vent pipe to relieve negative pressures. Perhaps in the next few seconds the situation would reverse. These vents were obviously unsuccessful in aerating the gate bottom, and gate vibration and bouncing prevailed.

Along with these vent pipe installations, five electric pressure cells, operating in conjunction with a 12-channel recording oscillograph, were installed on the centerline of one of the gate bottoms and 3 pressure cells were installed at each quarter point. At various tailwater elevations, pool elevations, and gate openings the instantaneous pressures on the 11 pressure cells were recorded simultaneously. Considerable difficulty was encountered in getting

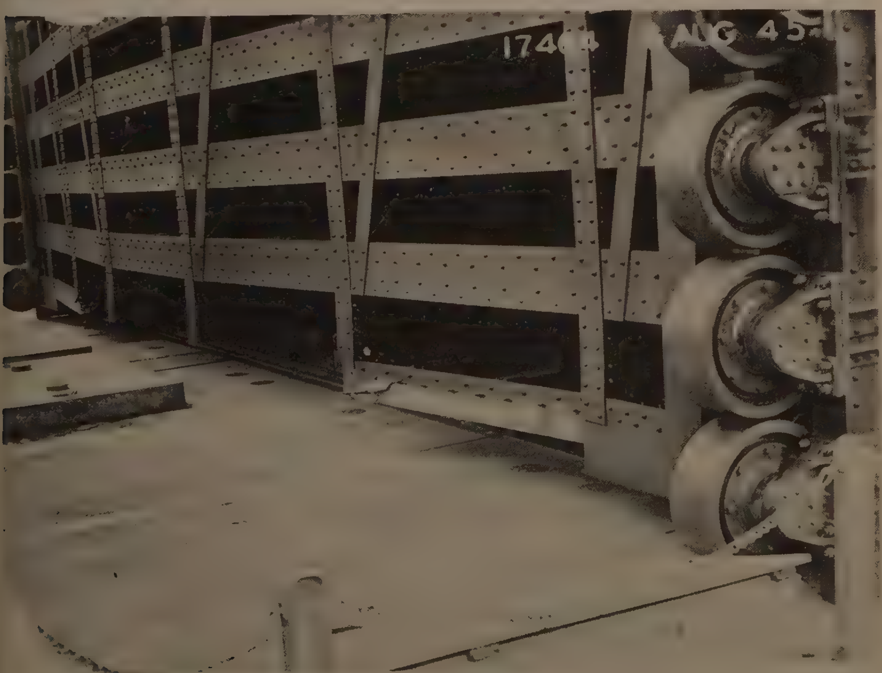


Fig. 6. Bonneville Spillway Gate Damage 1945

the pressure cells to function properly and the results were questioned in many cases due to improper action of the cells, but an overall picture of the complex pressure conditions on the bottoms of the gates was obtained from these tests. It was evident that pressures against the bottom of the gate upstream from the seal rapidly varied from positive to negative as the flow control shifted from the gate seal to the upper edge of the gate bottom as the gate opening approached some 3 feet. Alternate subjection of the gate to uplift and then down-pull as the jet sprang from one point to the other was believed to account for the gate vibration and bouncing. Also the fairwater plate area downstream from the seal had a high rate of pressure reversals, which varied from positive pressures at smaller gate openings to vapor pressure at larger openings. At a given gate opening, pressures at any one point were extremely variable with 75 reversals per second not uncommon. Comparison of conditions at the centerline of the gate with those at the quarter points showed no relationship. Negative pressures could exist at a centerline cell and positive pressures exist at the same time at a corresponding cell at the quarter-point or the reverse could hold true. Such erratic behavior was attributed to the alternate formation and collapse of vapor pockets between the submerged bottom of the gate and the water jet, modified by erratic effect of vortex action beneath the gate bottom and wave action against the downstream side of the gate.

Although no quantitative conclusions could be made from these tests, it was qualitatively determined that gate vibration occurred only when the gate bottoms were submerged, and the intensity of the vibration of the submerged gate bottoms was most severe at about a 3-foot gate opening. Fig. 7 depicts the relative intensity of vibration at various gate openings and tailwater elevations. This chart is now used by the operators to avoid gate settings which will give even moderate vibration. As a result no further gate damage has occurred. It has been concluded that it would take a major modification of the gate to eliminate the difficulty. It is considered cheaper to operate in such a manner as to avoid the conditions which sets up the trouble.

Stilling Basin Erosion

A number of papers and articles have been written in the past regarding the erosion of the stilling basin of the Bonneville spillway and the repairs which were made in 1955, and it is not intended to cover the details of this phase of the spillway operations in this paper. Fig. 2 shows a cross section of the stilling basin as constructed with a 63-foot long baffle deck at elevation -16 and 2 rows of trapezoidal shaped 6 x 6 foot baffles spaced 6 feet apart. By modern design standards this stilling basin is too short. Underwater surveys made in 1939, showed that erosion of the baffles and deck had developed even at this early stage. Model tests made at the Carnegie Institute of Technology in 1941 - 1942 indicated that the spillway gate openings should be kept as uniform as possible to avoid cavitational negative pressures on the baffles. Additional underwater surveys made in subsequent years showed continued deterioration of the baffles. Repairs to a few baffles in 1940 - 1941, modification in baffle shape as recommended by the Carnegie Institute of Technology tests, and even armoring the baffles with 3/8 inch steel plate proved to no avail. In 1954 model studies were made at the Bonneville Hydraulic Laboratory which demonstrated that stilling basin conditions and undermining of the apron downstream would become much worse if the baffles were allowed to deteriorate. Accordingly, the south half of the basin was unwatered in 1954 -

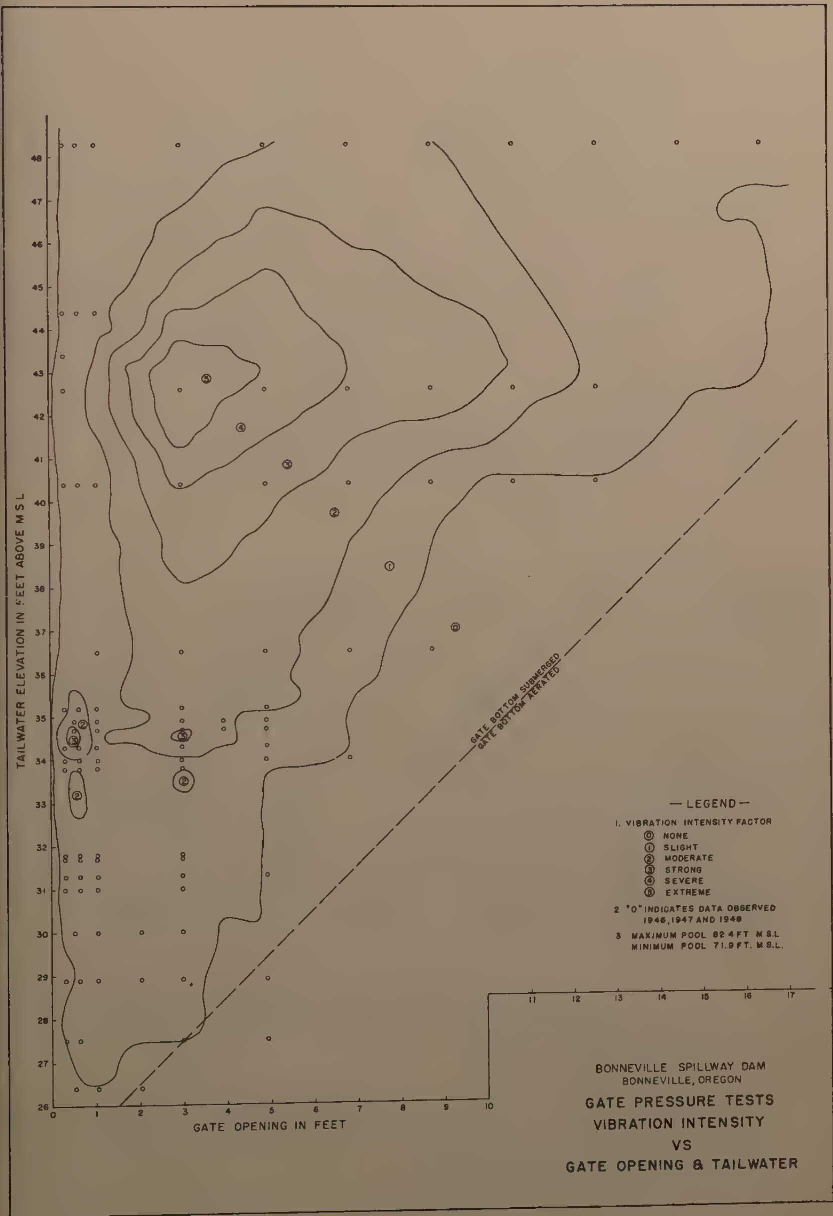

TAILWATER ELEVATION IN FEET ABOVE M S L
GATE OPENING IN FEET
GATE BOTTOM SUBMERGED
GATE BOTTOM AERATED
— LEGEND —
1. VIBRATION INTENSITY FACTOR
0 NONE
1 SLIGHT
2 MODERATE
3 STRONG
4 SEVERE
5 EXTREME
2 "O" INDICATES DATA OBSERVED 1946, 1947 AND 1948
3 MAXIMUM POOL 82.4 FT M S.L
MINIMUM POOL 71.9 FT. M S.L.
BONNEVILLE SPILLWAY DAM
BONNEVILLE, OREGON
GATE PRESSURE TESTS
VIBRATION INTENSITY
VS
GATE OPENING & TAILWATER

Figure 7

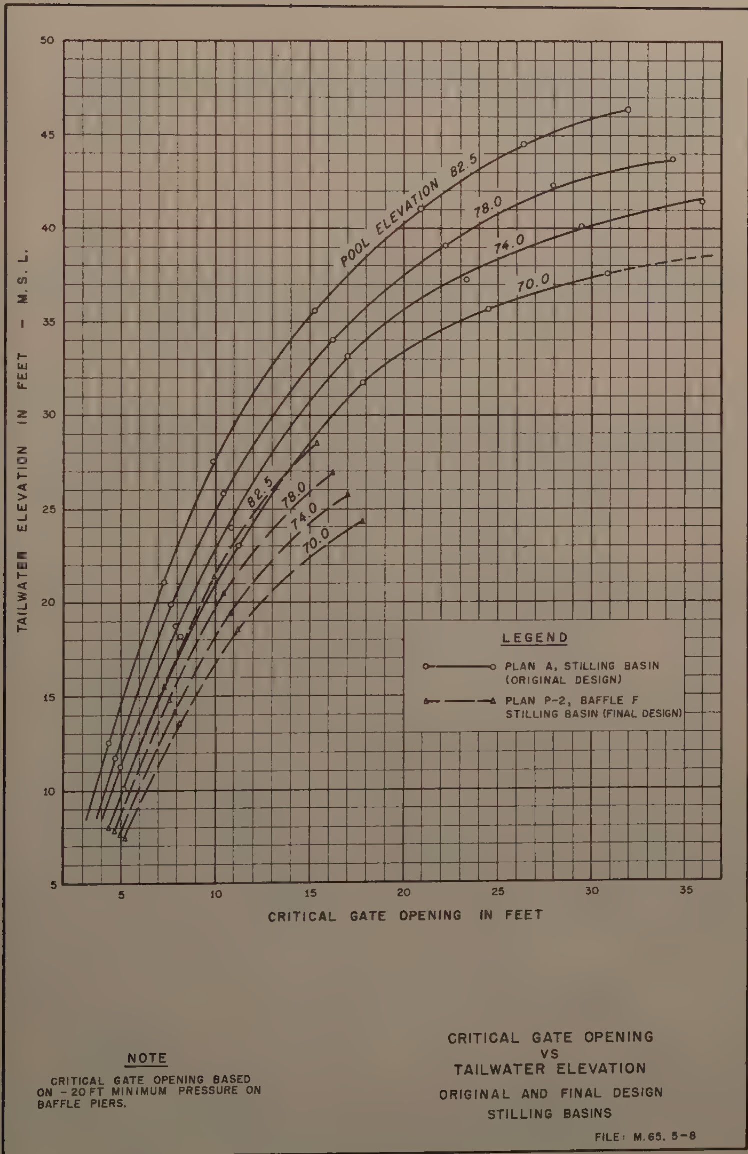
1955 and both baffles and deck were repaired. Based on Bonneville Hydraulic Laboratory tests the upstream row of baffles was streamlined, the downstream row of baffles was converted into a solid sill, and the deck level was raised one foot. The north half of the stilling basin has not yet been repaired.

Even with this repair, however, care must be taken to keep the spillway gate openings more or less uniform in order to distribute the energy evenly across the width of the stilling basin and prevent baffle cavitation. Critical gate openings have been established by model studies for various pools and tailwater elevations as shown in Fig. 8. Operation with gate openings greater than shown will more than likely produce baffle damage. The solid lines on this chart pertain to the original stilling basin which still exists in the north half of the basin; the dashed lines are for the revised basin on the south half. It will be noted that more leeway is permitted in the latter case. However, this places another restriction on the procedures which may be employed in operating the spillway. For a given pool and tailwater condition, the gates can neither be opened too much for fear of damaging the baffles (Fig. 8) nor too little for fear of damaging the gates (Fig. 7). In addition, the distribution of flow into the stilling basin must not interfere with the attraction flows to the fish ladders nor cause bank erosion downstream. Hence the operation of Bonneville spillway to maintain a given pool level is rather complicated.

Miscellaneous Operational Difficulties

The spillway gate wheels are designed to take uniform loading. They are mounted on the gate sections with heavy springs to take up any unevenness in the steel bearing plate in the slot. However, the rigid gate section does not permit too much flexibility and any one wheel can be stressed heavily if the bearing plate is only slightly uneven. Difficulty has been encountered with the roller bearings on the spillway gates at Bonneville Dam, particularly with the lower wheels on the bottom sections of the gates. The trouble was attributed to one of three causes: the shock transmitted to the wheel when pounding waves from the stilling basin struck the downstream side of the gate during high tailwater, bumping the lower wheel against the gate slot when introducing the gate into the slot with the gantry, or inadequate lubrication of the bearings under water. The roller bearings are barrel shaped, of special steel, and quite expensive. Cracked or shattered bearings were continuously having to be replaced. The problem is still current to some extent, but with care in operation, close maintenance, use of special water resistant greases, installation of grease line extensions to permit underwater lubrication, and an improved seal ring, the frequency of failures has diminished. The difficulty is somewhat inherent with this type of spillway gate, however, and it is doubted if it will ever be eliminated entirely.

Whenever a load rejection occurs at the powerhouse due to trouble on the transmission lines or other causes, the turbines are automatically shut down and a wave, sometimes 4 to 5 feet in height, travels up the pool and causes water to spill over the tops of the spillway gates. If the turbine flow is shut off for any length of time, the spillway gates must be opened to pass the equivalent amount of water. Closing down of 4 of the 10 powerhouse turbines for instance will cause the pool to rise 0.2 ft. per hour. The 2 gantries could each open one gate about 14 feet in a short time to take care of the situation, but from Fig. 8 it will be noted that this would cause cavitation at the baffles of the stilling basin if the tailwater was less than some 22 feet (240,000 c.f.s.).



TAILWATER ELEVATION IN FEET - M.S.L.
CRITICAL GATE OPENING IN FEET
POOL ELEVATION 82.5
78.0
74.0
70.0
82.5
78.0
74.0
70.0
LEGEND
PLAN A, STILLING BASIN (ORIGINAL DESIGN)
PLAN P-2, BAFFLE F STILLING BASIN (FINAL DESIGN)
NOTE
CRITICAL GATE OPENING BASED ON -20 FT MINIMUM PRESSURE ON BAFFLE PIERS.
CRITICAL GATE OPENING VS TAILWATER ELEVATION
ORIGINAL AND FINAL DESIGN STILLING BASINS
FILE: M.65. 5-8

Figure 8

Distribution of the outflow over a number of gates requires considerable time as pointed out previously, and the amount of freeboard and rate of pool rise may not allow adequate time for the operation. Therefore the usual procedur is to do nothing at the spillway to compensate for a load rejection at the powe house unless it becomes obvious that the outage will be extensive. If this is the case the distribution of flow over the spillway width is made as nearly uniform as time permits.

McNary Dam

McNary Dam is a multiple purpose project located on the Columbia River just upstream from Umatilla, Oregon and some 292 miles upstream from the mouth. It was the second dam built by the Corps of Engineers on the Columbi River. The overall length of the dam is 7300 feet, and it includes a concrete gravity spillway, a powerhouse with fourteen 70,000 KW units, a navigation lock and 2 fish-ladders. The total project cost is estimated to be $286,000,00 and it was put into operation in 1953. The normal pool is at elevation 340. A drawdown of 5 feet is made for power peaking purposes during low water whe necessary. The maximum head at the dam during low water is 92 feet.

Spillway

The 1310-foot spillway, located at the north end of the dam, has 22 bays with split vertical-lift gates 50 feet wide by some 52 feet high. The spillway ogee crest is at elevation 291. The 10-foot width piers support the roadway deck and the two 200-ton gantries. The spillway is designed to pass 2,200,00 c.f.s. at a pool level of 356.5 and 1,368,000 c.f.s. at normal pool level. The stilling basin is 248 feet long and has 2 rows of baffles 10 feet wide by 10.5 feet high and a 10-1/2-foot end sill. Tailwater varies from 248 at low water to some 273 at ordinary high water and 303 at design flood conditions. It should be pointed out that the crest of the spillway is above tailwater for all conditions where the spillway gates are used. In fact a tailwater of 291 is no reached until a river flow of some 1,368,000 c.f.s. prevails at which time all spillway gates are up and free flow exists over the spillway crest. This arrangement precludes any damage to the gate bottoms such as occurred at Bonneville Dam.

Hydraulically the McNary spillway functions quite satisfactorily. The flow conditions are similar to those predicted by the model studies and no difficulties of any consequence have developed thus far.

Spillway Gates

The spillway gates are in 2 sections with the bottom section some 24-1/2 feet in height. They are so designed as to pass flow between the two sections During low water conditions of course all flow passes through the powerhouse As the river discharge increases the spillway comes into use, and flow is passed between the two sections of the spillway gate with the bottom section on seal. The jet issuing between the gates in this manner is activated by a head of only about 27 feet which is not sufficient to create any acute vibration problems. Vents in the spillway piers aerate the underside of the nappe whe the spillway is operating in this manner. As the spillway flow further increa es, the two gate sections are joined and the spillway flow is passed beneath

the combined gate sections at the higher head of 50 feet similarly as done at Bonneville Dam. (See Fig. 10.) However, the duration of time that this latter method of operation is used is comparatively small. With some 200,000 c.f.s. going through the powerhouse and some 400,000 c.f.s. capable of being passed over the lower gate section, the undergate method of passing flow is not necessary except for approximately two to three per cent of the time. This arrangement has a distinct advantage over the Bonneville spillway gate where flow must be passed beneath the gates about 60% of the time.

The lower section of the spillway gates at McNary Dam is articulated in such a manner as to permit flexibility in transmitting the gate thrust to the wheels. The section is in three panels each with two wheels on a side to support the thrust. Thus, any unevenness in the wheel track can be followed by the wheels with no change in load. This is an improvement over the rigid section as used at Bonneville Dam, and contributes to less maintenance of the wheel bearings.

Spillway Gate Slot

Based on the difficulties experienced at Bonneville Dam, the gate slot at the McNary spillway was made as narrow as possible and the downstream corner was beveled to eliminate gate slot damage. The width of only 2-1/3 feet versus the 7-foot width at Bonneville offers less opportunity for the jet to spread and strike the downstream end of the slot. The bevel, consisting of a 10-foot radius over a 3-foot length set the downstream corner of the slot

Fig. 9. McNary Spillway—Split gate flow

back by almost 4 inches. These improvements apparently are fairly successful since operation of the spillway to date has shown only a small amount of pitting on the ogee section near the corner of the gates.

Stilling Basin

With the stilling basin designed for a hydraulic jump at the maximum flood of 2,200,000 c.f.s., there has been no difficulty with its operation at the normal floods which have been experienced since the project was put into operation. As a matter of fact, with its ample length and depth and stream-lined baffles, no difficulty is anticipated.

Load Rejection

Unlike Bonneville Dam which has a relatively narrow pool upstream from the dam, a load rejection of all 14 power units at McNary Dam only causes a wave of some 6 inches due to the wide pool upstream from the dam. Furthermore the area of pool is about twice that of Bonneville Dam and hence impoundage of the same amount of flow at McNary Dam causes the pool to rise only half as fast. This is a distinct advantage in spillway operation since the gantry cranes have twice the time to correct the spillway gate openings if the powerhouse outage is prolonged.

Fig. 10. Under-gate flow at McNary Spillway

Chief Joseph Dam

Chief Joseph Dam is located on the Columbia River near Bridgeport, Washington, approximately 51 miles downstream from Grand Coulee Dam and 546 miles above the mouth of the river. It is the third Corps of Engineers dam to be built on the main stem of the Columbia River. The total cost is estimated at $160,000,000. The construction of the dam, including the spillway, was completed in 1955 and the installation of the initial sixteen 64,000 KW powerhouse units is to be completed this year. The dam is multipurpose with power as its chief direct benefit. The principal features of the dam are a 922-foot concrete gravity spillway, an intake structure for the ultimate 27 powerhouse units, and the powerhouse for the initial 16 units. The maximum design flood is 1,250,000 c.f.s., the maximum flood of record is 740,000 c.f.s., and the normal flood each year is about 300,000 to 400,000 c.f.s. Normal low water flow is about 50,000 c.f.s. The head at the dam is about 165 feet.

Spillway and Gates

The height of the spillway from the crest elevation of 901.5 to the floor of the stilling basin is 158.5 feet. Atop the crest of the spillway are nineteen 42-foot-radius tainter gates 40 feet wide and 49 feet high. These gates are operated by individual gate hoists through a sprocket and chain arrangement. The trunnions and the bottoms of the gates, when wide open, are clear of the maximum flow nappe. Normal pool is at 937.5, maximum pool is at 955 with the design discharge and free flow over the spillway crest. Fig. 11 shows the spillway in operation.

Fig. 11. Chief Joseph Spillway in Operation

Operation experience at the spillway to date has been very limited since it has only been in use some 3 years. However no major difficulties have developed so far. Flow conditions in the spillway are in line with model study results. The gates are simple to operate by individual push-button controls and have limit switches at gate-opening intervals to prevent the gate from being accidently opened too far. The ease with which these spillway gates can be handled has led to the decision to install necessary equipment to operate them by remote control from the powerhouse control room. Heaters have been provided in the gates to offset icing, but to date no appreciable ice conditions have occurred which would require their extensive use. Debris can readily be passed beneath any of the gates and this offers no problem. The absence of gate slots eliminates gate-slot trouble. Maintenance on these tainter gates is minor and operation manpower is limited to a maximum of one man. All in all the operation of these spillway gates at Chief Joseph Dam is simplicity itself in comparison to the spillway gate operation at Bonneville Dam.

Stilling Basin

The stilling basin at the Chief Joseph spillway is 167 feet long. A single row of streamlined "Bluestone" baffle piers 11 feet high, 13-1/4 feet wide and 22 feet long is located on the 743 deck some 58 feet upstream from the lower end of the basin. A stepped 11-foot high end sill is located at the downstream end of the stilling basin. (See Fig. 12). Satisfactory hydraulic jump condition have prevailed with all flows passed through the spillway to date, but an unde water survey of the basin in 1957 revealed some erosion of the baffles and deck of the stilling basin.

The Dalles Dam

The Dalles Dam is located on the Columbia River near the City of The Dalles, Oregon, some 192 miles upstream from the mouth of the river and about 47 miles upstream from Bonneville Dam. It is the fourth dam to be constructed by the Corps of Engineers on the Columbia River. The dam is

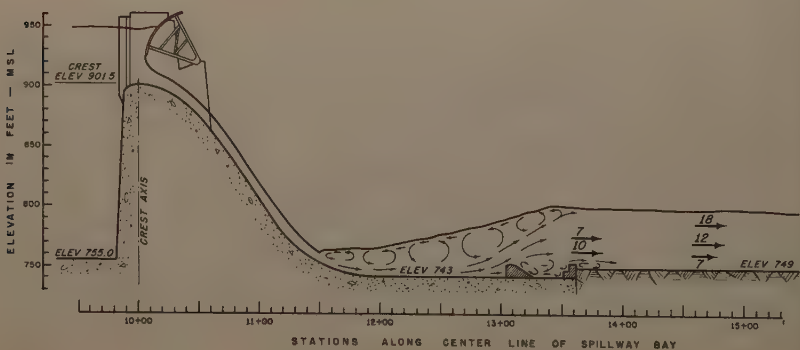


Figure 12
Spillway Cross Section at Chief Joseph Dam

8700 feet long, will cost $250,000,000 and consists of a single-lift barge lock, a 23-bay spillway, a powerhouse with fourteen 78,000 KW units initial installation (22 units ultimately), a rock-fill closure dam and fish passage facilities including 2 fishladders. The project was designed for a maximum flood of 2,290,000 c.f.s.; the flood of record at this site was 1,240,000 c.f.s.; normal high water is about 575,000 c.f.s.; and low water flow ranges from 50,000 to 100,000 c.f.s. Normal pool is at elevation 160 with provisions for a 5-foot drawdown for power peaking purposes. Normal head at the dam is 87.5 feet. Construction of the dam was started in 1952, closure of the dam was accomplished in 1957 and all 14 initial powerhouse units will be installed by 1960.

Spillway and Gates

The 1370 foot spillway at The Dalles Dam is shown in operation in Fig. 13 during the fall of 1957. The crest of the spillway is at elevation 121. The spillway is designed to pass a design flood of 2,290,000 c.f.s. at a pool of 182.3 and a project flood of record (regulated) at normal pool of 160 with all 23 spillway gates open. The spillway tainter gates are 50 feet wide and 41 feet high and supported by 4 cables on each corner. Each gate has its individual gate hoist which raises or lowers the gate at a speed of about 1 foot per minute. Limit switches stop the gate movement at one foot intervals after each "push button" start. Under ordinary flow conditions, all gates can be closed to seal in about 10 minutes if necessary. The height of the spillway gates was dictated by the depth to which downstream-bound fingerling salmon could be submerged and then released to atmosphere pressures as they passed beneath the gate. After the 14 powerhouse units are in operation in 1960, all flows of about 200,000 c.f.s. and less will be passed through the powerhouse. This will reduce the time in which the spillway will be in operation to about 4 months per year, rather than 100 per cent of the time as at present.

Operation experience with The Dalles spillway gates has to date been entirely satisfactory. There was a little difficulty initially in getting a balanced load on the cables supporting the gates, but once this was adjusted, the gate operating machinery and supporting cables have functioned without mishap. There has been no ice to date but heaters have been provided for this condition when it occurs. Debris has been no problem. Some freezing of the spray has given slight inconvenience but nothing more. The gates can be adjusted easily with the aid of gate opening indicators and revolution counters on the gate actuating shafts, and pool levels are easily maintained to 0.1 foot of the desired level. The hydraulic conditions appear to be in accordance with model study predictions. Again, as at Chief Joseph Dam, the absence of gate slots and non-submergence of the gate bottoms at high water as occur at Bonneville Dam has made The Dalles spillway operation and maintenance relatively simple.

Stilling Basin

The Dalles stilling basin is 170 feet long, and has one row of 9-foot "Bluestone"-type baffles backed up by a 13-foot vertical end sill. (See Fig. 14). This relatively high end sill was primarily to reduce rock excavation on the apron downstream from the stilling basin. Although The Dalles Dam is at the head waters of the Bonneville Dam pool and The Dalles tailwater is basically affected thereby, the relatively high end sill and apron of The Dalles stilling

basin create a control on The Dalles stilling basin outflow at flows of less than 400,000 c.f.s. The stilling basin is designed to create a hydraulic jump for all discharges up to the flood of 2,290,000 c.f.s. However, the criteria was adopted in the design of the stilling basin that no greater depth or length would be provided than was absolutely necessary to give a jump in the stilling basin with even distribution of flow across the width of the stilling basin. No allowance was made for possible uneven distribution. This was further restricted by the fact that only 20 of the 23 bays would be available for flow passage up to river flows of 800,000 c.f.s. since 2 bays on one end and one bay on the other end would be restricted to small outflows for fish attraction purposes at those lower flows. The stilling basin as constructed is therefore rather critical, and little leeway is afforded in its operation. With the rather restrictive stilling basin considerable care must be taken to keep the spillway gates at uniform opening. This is not too difficult with the easily manipulated tainter gates on the spillway, but it creates difficulties when gates are taken out for maintenance or it is necessary to open up one or two gates for debris passage.

Another situation which places restrictions on the method of spillway gate operation, is the fact that at high water the high velocities and wave action created in the stilling basin carry on across the stilling basin apron and the shallow reefs downstream from the spillway, and attack the piers of the bridge located some 2000 feet below. This same wave action also attacks the gravel guide wall fill downstream from the navigation lock. This demands again that the spillway gate openings be as uniform as possible. There has been no attempt to measure the prototype velocities across the rock apron downstream

Fig. 13. The Dalles Spillway in operation—1957

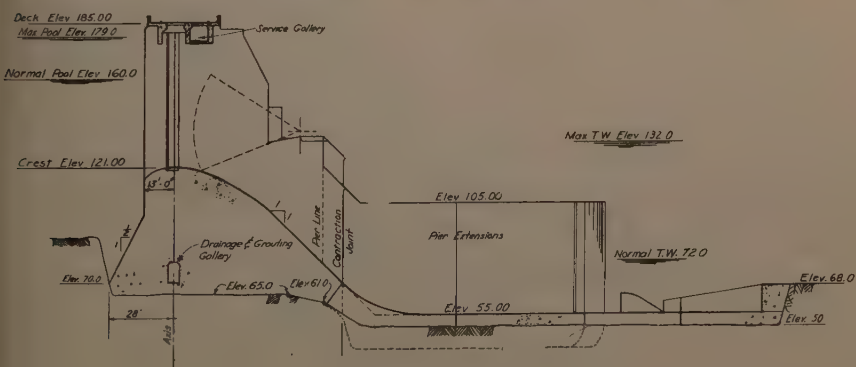
from the stilling basin, but they are relatively high and may cause some erosion of that apron. By the summer of 1959, a sufficient number of powerhouse units will be on the line so that the spillway can be closed and an underwater survey made to determine the condition on this apron.

Detroit Project

The Detroit Project, constructed by the Corps of Engineers in 1949 - 1954 at a cost of $63,000,000, consists of Detroit Dam and its downstream reregulating Big Cliff Dam, located on the North Santiam River, a tributary of the Willamette River, some 45 miles east of Salem, Oregon. The primary purpose of the Detroit Project is flood control with power as an important secondary function. Detroit Dam is a 1522-foot long concrete gravity type structure, some 463 feet high from foundation to deck and normally operates with a head of from 250 to 360 feet. The pool is drawn down some 113 feet during the rainy winter season to provide about 300,000 acre feet of flood control storage; during the drier summer months it is maintained at the normal pool elevation of 1569. The two 50,000 KW powerhouse units at Detroit Dam are used primarily for peak power. The average flow of the North Santiam River is 1764 c.f.s., the maximum flow of record is about 65,000 c.f.s., and the design flood is 176,000 c.f.s. The reregulating Big Cliff Dam is located 3 miles downstream from Detroit Dam and its purpose is to impound the irregular outflow from Detroit Dam, caused by power peaking operations, and pass it downstream at a uniform rate. The 18,000 KW Big Cliff powerhouse unit is remotely controlled from the Detroit powerhouse control room.

Spillway of Detroit Dam

The Detroit Dam spillway is designed for 176,000 c.f.s. with all 6 spillway gates wide open. The crest elevation is at 1541. (See Fig. 15). The tainter gates atop the spillway crest are 42 feet wide and 31 feet high. Each gate is operated by individual gate machinery beneath the spillway deck. The gates are supported by cables attached to each bottom corner of the gate.



SECTION B-B
Scale: 1" = 20'-0"

Figure 14
The Dalles Spillway cross section

"Eyebrows" were constructed in the spillway chute just above the conduit openings to cause the spillway flow to jump across the openings and not impinge on the floors of the conduits, and these "eyebrows" function quite satisfactorily. The Detroit spillway has been in operation for limited periods of time each year since its construction (See Fig. 17), and it is not in continuous use over long periods of time as in the spillways of the Columbia River dams. To date the only problem at Detroit Dam has been the minor one of leakage past the rubber seals in the corners of the gates when the gates are closed. Hydraulically the spillway performs quite satisfactorily when in operation.

When the Detroit spillway was initially constructed, that portion of the chute beneath the right spillway bay was separated from the chute below the other 5 bays by a concrete training wall as shown in Fig. 17. This section is locally called the "test chute". Its purpose is to permit study of high velocity flow down high spillway chutes, particularly with regard to air entrainment and consequent bulking of flow. The chute was contained within parallel walls to prevent lateral spread of the flow as it traveled down the chute. Installed within one chute wall are windows for observing the cross sectional characteristics of flow in the chute. Electrodes are also installed in the floor and walls of the test chute to measure velocities of flow. A carriage extending across the chute has instrumentation for measuring water surface elevations. Although flow has been passed down the chute, no full scale test program has as yet been implemented. All facilities are on hand for making comprehensive tests as may be required, however.

The stilling basin of Detroit Dam spillway is about 170 feet long, has 2 rows of 6-foot baffle piers and a stepped end sill. The deck of the stilling basin is designed to create a hydraulic jump at the design discharge of

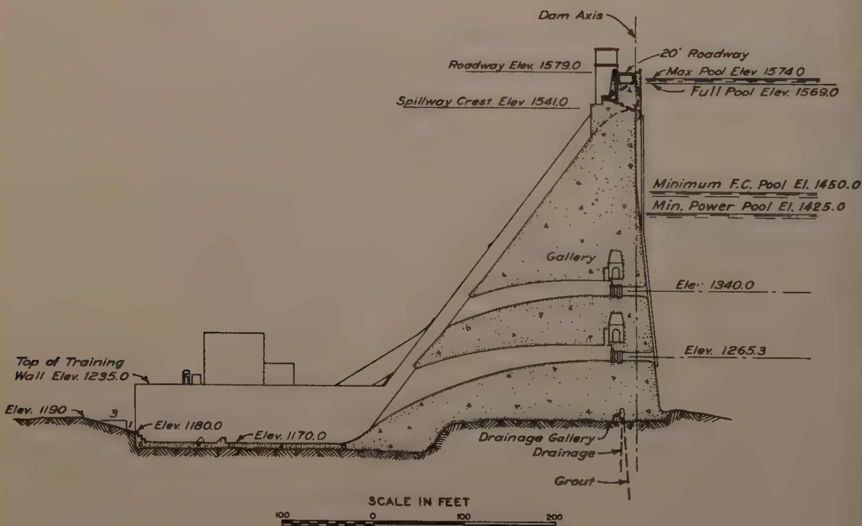


Figure 15
Detroit Spillway Cross Section

176,000 c.f.s. No difficulties have been encountered thus far in its operation either with flow over the spillway or through the conduits.

Detroit Conduits

There are 4 flood control conduits in the Detroit spillway—two are at the lower level of 1265 and two are at the upper level of 1340. All four are controlled by vertical-lift hydraulically operated slide valves 5-2/3 x 10 feet in size. The conduits are designed for open-channel flow downstream from the slide valves. The conduit height is abruptly increased from 10 feet to 16 feet in height after it passes the valve to prevent the conduit from flowing full. A 2-1/2 foot diameter air vent enters the roof of the conduit at this point. The width of the conduit is also increased from 5-2/3 feet to 7 feet at the slide valve to prevent the water jet from striking the downstream side of the gate slot. The width of the conduit between the service valve and the emergency valve (8 feet upstream) was tapered by 3-1/2 inches and steel lined to prevent slot cavitation. No steel lining was placed downstream from the service valve. The maximum head on the lower-level conduits is some 304 feet; the head on the upper level conduits is about 230 feet. The design was based on use of the conduits when the pool was below the spillway crest level, but the conduits have been used during the higher pool levels with the spillway gates closed. When the conduit valves were closed, uplift pressures on the 45 degree beveled valve bottom caused it to open, and it was necessary to put pressure on the hydraulic oil cylinder at all times to prevent this action.

After the Detroit conduits had been in operation a short while, it was noted that the concrete floor and lower portion of the side walls of the lower-level conduits had been eroded extensively. The concrete had been cut back some 12 inches for a distance of 30 feet downstream from the valves, and the embedded reinforcing steel had been exposed or in some cases torn out. (See

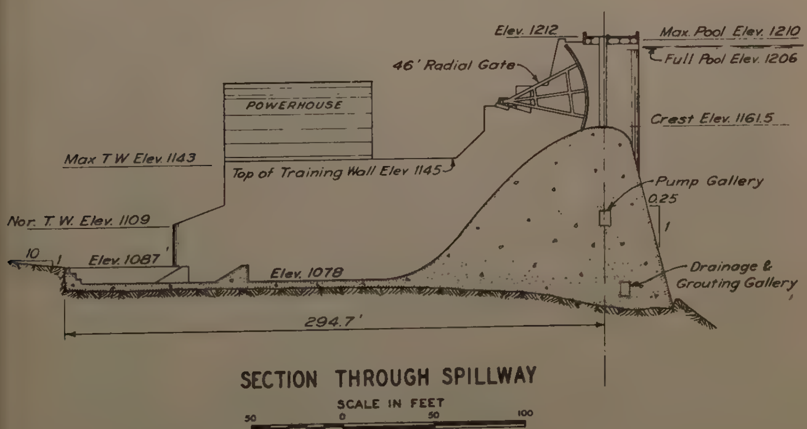


Figure 16
Spillway Cross Section - Big Cliff Dam

Fig. 18) Some slight erosion had occurred in the upper-level conduits, but nothing in comparison to the lower-level conduits. The model studies of these conduits had indicated no negative pressures in the eroded area and readings from the piezometers which had been installed in one of the prototype conduits had confirmed the model data. It appeared that the damage to the concrete was caused initially by erosion of the comparatively weak mortar at the horizontal pour lines, followed by local cavitation after the large aggregate was exposed. It was decided to repair one of the lower-level conduits and both upper-level conduits by high strength mortar and concrete. This was done and the repaired conduits subjected to flow conditions again. The repairs on the upper-level conduit were satisfactory and it was concluded that these conduits need not be further repaired although it was contemplated that repairs might again be necessary in 5 to 10 years. It was apparent, however, that the high-strength concrete could not withstand the abrasion of the conduit jet in the lower-level conduits, since not only were the repair patches torn out, but additional parent concrete was eroded. It was decided to steel line these lower-level conduits. This was done and no further trouble has developed. From

Fig. 17. Detroit Spillway in operation

this experience and the operation of the conduits in general, it would appear that the conduits would have been quite satisfactory if they had been steel lined initially. The conduit itself functions properly and no trouble has developed with the slide gate even though it is operating under heads of 300 feet.

An 8-foot-diameter test conduit was installed on the south side of the spillway in Detroit Dam during the initial construction. This conduit is gated at the upper end, and is to be used for full scale tests of such items as valves, conduits, or other devices. The test conduit will pass 4,000 c.f.s. at a head of about 230 feet. To date it has not been used, but a test program is being developed.

Big Cliff Spillway

Fig. 16 shows a cross section of the Big Cliff reregulating dam. It has an operating head of 97 feet. It is a concrete-gravity structure with three 46 feet high by 44-1/2 feet wide tainter gates capable of passing the Detroit Project design flood of 176,000 c.f.s. The gates are activated manually by "push button" controls and gate operating machinery on the spillway deck. Cables are used to support the gates in an open position. One of the gates is automatically opened if the Big Cliff powerhouse unit is shut down for load rejection or other causes. Some thought is being given to operating all of the spillway gates by remote control from Detroit Dam. To date no major difficulties, either hydraulically or structurally, have developed with the operation of the Big Cliff spillway or stilling basin.

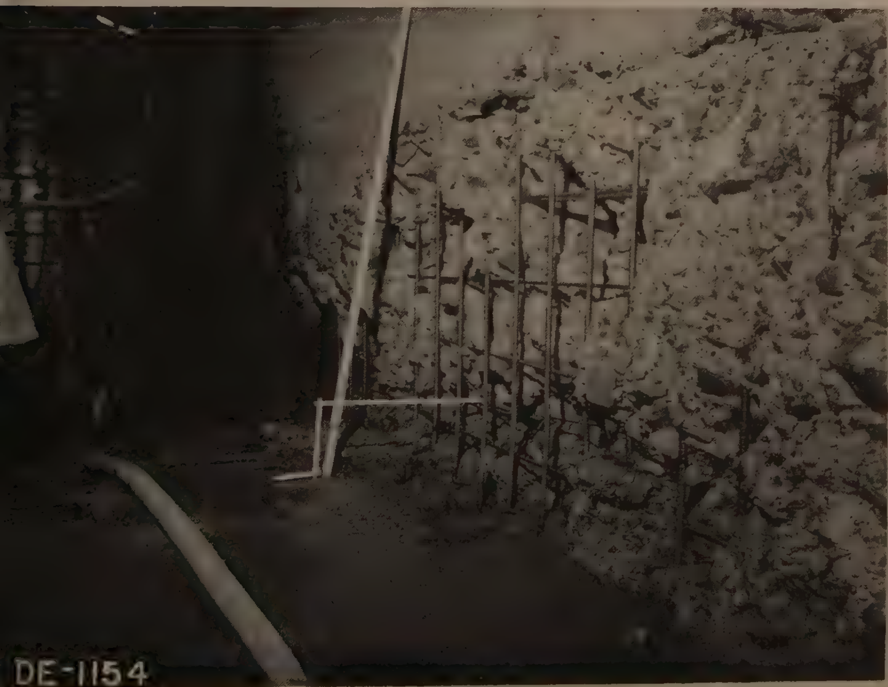


Fig. 18. Eroded Conduit in Detroit Spillway

Lookout Point Project

Lookout Point Project, consisting of Lookout Point Dam and its reregulating Dexter Dam, was constructed by the Corps of Engineers in 1949-1955 at a cost of $88,000,000. The project is located on the Middle Fork of the Willamette River some 21 miles east of Eugene, Oregon. Lookout Point Dam consists of an earth-fill dam combined with a concrete gravity spillway structure some 258 feet high. Its primary purpose is flood control with powe its principal secondary purpose. As at Detroit Dam, the pool is drawn down some 104 feet in the winter rainy season to provide 337,000 acre feet of flood control storage. The design discharge is 270,000 c.f.s., the flood of record i 87,000 c.f.s. and the low water flow is about 1000 c.f.s. Lookout Point Dam has already experienced a flood of very close to record magnitude. It was successful in withholding the entire flood within its reservoir and reducing the flood level at Eugene by some 10 feet. The powerhouse located at Lookou Point Dam has three 40,000 KW units which are used primarily for peak power. The head at Lookout Point varies from 235 feet in summer to 130 fee in winter. Dexter Dam some 3 miles downstream reregulates the varying ou flow from the Lookout Point turbines by daily ponding. Dexter Dam is also a earth-fill concrete gravity spillway dam with a head of 55 feet and a single powerhouse 15,000 KW unit remotely controlled from Lookout Point powerhouse.

Lookout Point Spillway

The spillway at Lookout Point Dam has 5 bays capable of passing a discharge of 270,000 c.f.s. (See Figs. 19 and 21). Control of the spillway flow i by 42-1/2 foot wide tainter gates mounted on top of the elevation 887.5 feet m.s.l. ogee crest. These gates are supported by cables from individual

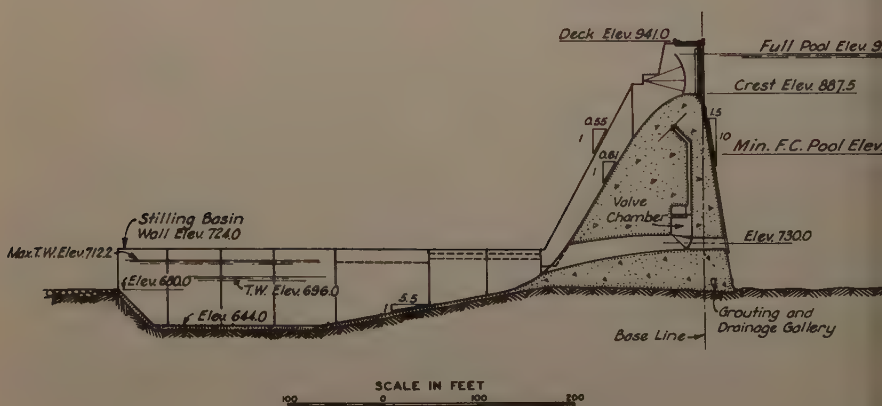


Figure 19
Spillway Cross Section at Lookout Point Dam

gate-hoisting machinery beneath the spillway roadway deck. On the chute section of the spillway, 2-foot high "eyebrows" (Fig. 22) deflect the flow and cause it to jump the conduit openings. The stilling basin below the spillway has a sloping floor, no baffles, and 1 on 1 sloping end sill. It is designed for a hydraulic jump at 175,000 c.f.s. To date the spillway and stilling basin have operated quite satisfactorily with the small flows passed through them and no difficulties have developed with the tainter gates, spillway crest, chute or stilling basin.

Conduits at Lookout Point

There are 4 gate controlled conduits 6-3/4 feet wide by 12 feet high and operating under a head of 95 feet to 200 feet in the spillway of Lookout Point Dam. These conduits were designed to pass flow when the pool level was below the spillway ogee crest or a maximum of 158 feet head, but they have been used up to the 200 foot head. They are designed for open-channel flow downstream from the 20-foot radius tainter control valve (Walker valve). The roof of the conduit is raised 4 feet after it passes the valve and the conduit width is increased from 6-3/4 feet to 8-1/2 feet. The valve trunnion operates on a cam to seal the valve tightly, but a great deal of undesirable spray occurs in the valve chamber when it is pulled off of the seal during valve movement. Although the conduits are vented downstream from the valve, and sufficient roof space is available for air to enter from the downstream end of the conduit, (See Fig. 22) there is a definite pulsating in the valve chamber. Measurements have indicated that the negative pressure is only a few ounces below atmosphere but it nevertheless is indicative of slight unstability. To date the valve has been opened wide at a head of 150 feet, and to 0.7 opening at full 200-feet head with no tendency to develop full-tunnel flow conditions.

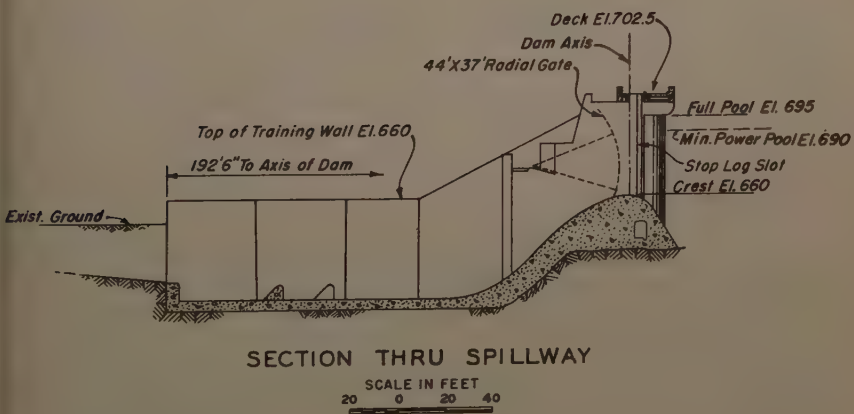


Figure 20
Spillway Cross Section at Dexter Dam

Erosion of the concrete in the conduits has been watched carefully. Although some slight sandy roughness of the walls has developed, there has been no tendency to erode in the same manner as at Detroit Dam.

Spillway at Dexter Dam

This spillway is a concrete gravity structure with seven 35 feet wide by 44 feet high tainter gates mounted on the ogee crest at elevation 660. (See Fig. 20). These gates are also raised by cables from individual hoists. The stilling basin downstream from the spillway has 2 rows of baffles and a vertical end sill. Both the spillway and stilling basin are designed to pass a flow of 270,000 c.f.s. Hydraulically the Dexter spillway has functioned quite satisfactorily thus far. Crystallization of the 2-inch diameter bolts attaching the cables to the spillway gates occurred in the early stages of operation, and unintentionally non-free slide plates beneath the gate trunnion arms caused the ends of several of the piers to crack, but both of these minor structural deficiencies were easily corrected.

Other Northwest Dams

Pelton Dam

Pelton Dam is a 200-foot high arch dam recently constructed by the Portland General Electric Company on the Deschutes River, a tributary of the Columbia River, near Madras, Oregon. It has a 3 unit, 120,000 KW capacity

Fig. 21. Flow over Lookout Point Spillway

powerhouse and a 75-feet wide, 30,000 c.f.s. capacity "ski-jump" type of spillway controlled by two 34 x 22 foot tainter valves on the left abutment. The chute portion of the "ski-jump" spillway is concrete lined. The flip at the lower end of the chute causes the flow to jump clear and plunge into the lower pool. Operation of this spillway creates a great deal of spray in the canyon but otherwise it appears to function quite satisfactorily for this type of dam. It is not intended that it will be used frequently.

Swift Dam

Swift Dam now under construction by the Pacific Power and Light Company is 45 miles northeast of Portland and is the third dam is a series of three power dams on the Lewis River, a tributary of the Columbia River. It is an earth-fill dam some 500 feet high. It will have three 68,000 KW powerhouse units. The spillway is a concrete lined chute in the rock cliff on the left abutment, and carries the flow some 1750 feet downstream from the control gates before it is flipped into the river bed downstream.

Cougar Dam

This 445-foot-high dam is now under construction by the Corps of Engineers. It is located on the South Fork of the McKenzie River, a tributary of the Willamette River. The dam site is some 44 miles east of Eugene, Oregon. The project will consist of a rock-fill dam with an earth core, a gated chute-spillway and a powerhouse with two 12,500 KW units. The flood control storage of the dam is about 155,000 acre feet. The 80,000 c.f.s. spillway on

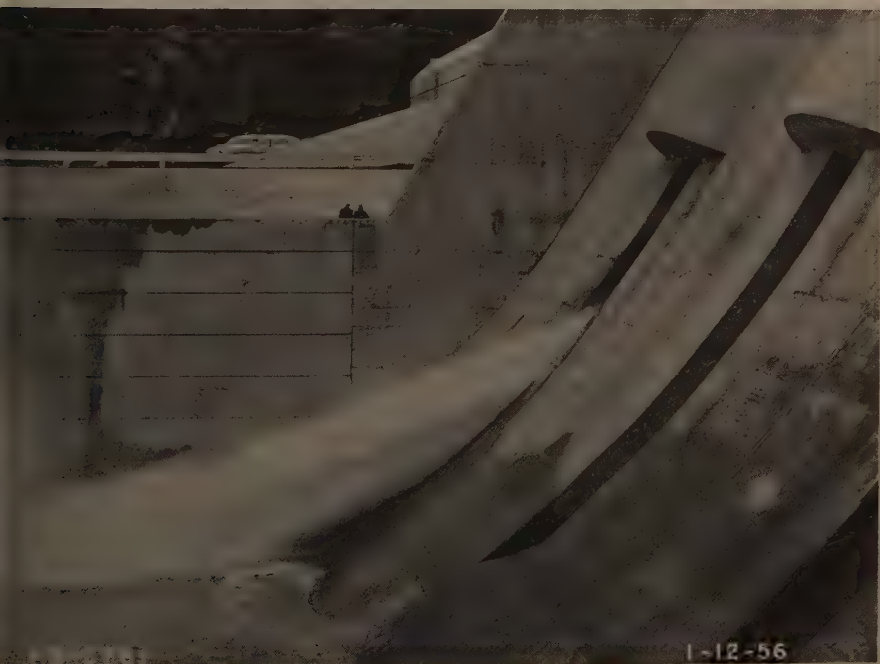


Fig. 22. Lookout Point Conduit in Operation

the right abutment has an approach channel, two tainter gates for controlling the flow, and a concrete chute down the hillside with no stilling basin. This type of spillway was adopted for this dam since it will be used only infrequently.

Hills Creek Dam

This dam is also being constructed by the Corps of Engineers at the present time. It is located on the Middle Fork of the Willamette River 26.5 miles upstream from Lookout Point Dam. It will be a flood control dam with 200,000 acre feet of flood control storage and two 15,000 KW power units oper ating under a maximum head of some 317 feet. The units will be remotely controlled from Lookout Point Dam. The dam will be of an earth and gravel filled type, and have a 328-foot-long concrete lined chute-type spillway on the right abutment. The chute will converge in width, be on a 30 degree slope anc have a flip bucket at the bottom. Three 42 foot x 48 foot gates will control the spillway flow at the head of the chute. As at Cougar Dam it is not expected that the Hills Creek spillway will be used to any great extent.

It will be noted that at all four of these modern dams, the spillway is of the chut-flip type. They are located in the abutment and spill into the stream bed below with no stilling basin. For dams of this type located in narrow valleys with smaller drainage basins and small flood runoffs, the chute-flip type of spillway is not only more economical to construct, but with its infrequent use should not present any major operation problems.

Vertical-Lift Gates Versus Tainter Gates

This paper has described the operating experience with two types of spillway control gates—the vertical-lift gates of Bonneville and McNary Dams, an the tainter gates of Chief Joseph-, The Dalles-, Detroit-, and Lookout Point Dams. Comparison of this experience indicates that the tainter gate has a number of advantages over the vertical-lift gate when operated by gantry cranes. The rapidity with which a single gate of either type can be opened or closed is not too different, but with the usual need to keep gate openings uniform across the spillway, it is normally necessary to open or close a numbe of gates at one time, and here the individually operated tainter gate is far superior to the slow-moving gantry cranes. All gates on the spillway at The Dalles Dam for instance can be entirely closed in about 15 minutes even with high flows; this same operation at Bonneville would take 4 to 5 hours with 2 gantry cranes operating.

The latching or dogging arrangement on the vertical-lift gates restricts th selected change in opening to 1 latch or "dog" which may be one or two feet apart. With the tainter gate the spillway gate can easily be set to 0.1 feet. This gives more flexibility and facilitates spillway operation.

One of the biggest advantages of the tainter gates is the elimination of the vertical-lift gate slots with their attendant cavitation. Such gate slots can be reinforced with steel and can be beveled to reduce the attack of the water jet but complete elimination of the slot altogether assures reduced maintenance.

The manpower required to operate the vertical-lift gate in combination with a gantry is greater than that required to operate tainter gates which are individually operated by push button controls and individual hoisting machine This reduces operating costs. The relatively simple machinery needed to

operate the tainter gate lends itself more readily to remote control and further reduction in manpower operating costs. This same simplicity also reduces maintenance costs.

CONCLUSIONS

From an operation standpoint, spillways and other features of dams should be so designed as to minimize operation and maintenance costs. This is often overlooked in design. Economy in construction cost is of course desirable, but if this economy is to be gained by increased manpower to operate and maintain the facility, the original economy is nullified. A saving of one man on the operations payroll is worth $100,000 in initial construction cost. Based on the operating experience with spillways in the Northwest dams as outlined in this paper, the following conclusions can be drawn:

Individually operated tainter gates are superior to vertical-lift type of gates operated by a gantry crane, and lend themselves more readily to remote control.

If vertical-lift type of gates must be used, the spillway crest should be kept above flood tailwater, a sharp bottom shape should be provided on the gates, and a split gate arrangement should be incorporated if possible.

Dependable limit switches should be installed both on tainter gates, and gantry cranes which lift vertical-lift type gates, to prevent improper operation and expensive repairs.

Use of cables to support tainter gates in the open position seems to be just as satisfactory as the chain and sprocket equipment.

Stilling basins which are subject to year-around use should be of ample proportions to avoid restrictive operations and expensive maintenance.

Chute-flip type of spillways on high head dams can be advantageously used when the use of the spillway is infrequent and the spillway capacity is relatively small.

High-head outlet conduits should be steel lined for a distance downstream from the valve to prevent erosion of the concrete by the high velocity jet.

The advantage of the open-channel type of flow in high-head outlets is questionable.

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CONSISTENCY IN UNITGRAPHS

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SYNOPSIS

The successful application of the unitgraph method calls for both experience and judgment. Because judgment is based partly upon preconceived ideas, unitgraphs that are derived by visual inspection are influenced by those ideas and in turn, help to perpetuate them. For a firmer understanding of unitgraphs, more arithmetic should enter into their determination and greater use should be made of compound flood events as sources of data. This paper undertakes a re-appraisal of certain common assumptions and presents a rapid and positive arithmetical method of deriving unitgraphs from compound hydrographs. An urgent need for more study of lag relationships is demonstrated.

INTRODUCTION

The runoff hydrograph is the end result of complex natural phenomena that cannot readily be expressed in mathematical terms. In the unitgraph method(1) a simplified approximation of those phenomena is employed to reconstruct the hydrograph, by the use of certain working assumptions. Further progress in unitgraph procedures would be difficult if there were unqualified acceptance of the assumptions that are in general use. Among those that are widely accepted, some of them as physical truths, the following should now be reviewed critically:

a. That the unitgraph represents "surface runoff".
b. That the total volume of the unitgraph must be equal to one standard unit.
c. That all unitgraphs for a given river point have the same length of base.

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d. That the unitgraph must end with zero discharge.
e. That the recession limb of the unitgraph, beyond the point of inflection (cartesian plotting), represents "discharge from channel storage".
f. That a unitgraph and a series of excess rainfall increments are necessarily correct if, when used together, they will reproduce the hydrograph of direct runoff.
g. That all unitgraphs for a given river point have the same lag time.
h. In addition to the above, there is the basic principle of the unitgraph method itself, which for certain purposes has to be accepted: that all unitgraphs for a given river point are mutually proportional in shape, with respect to their ordinates.

The validity of assumption (a) depends upon what is meant by "surface runoff". The three elements of streamflow that can be recognized in the discharge hydrograph are ground-water flow, interflow, and overland flow.(2) Ground-water flow is the characteristic discharge from deep ground storage and it has a long concentration time, depending in part upon the time required for rainwater to percolate down to the ground-water level. Interflow consists essentially of water that has infiltrated to shallow depths before entering the channel.(3) It includes subsurface flow, discharge from bank storage, and minor flows from unstable ground-water slopes near the river, all with a concentration much faster than that of ground-water flow and considerably slower than overland flow. The latter consists of water that has not entered the ground, and includes channel precipitation.

When L. K. Sherman first presented the unitgraph in 1932, only ground-water flow and surface runoff were recognized. Interflow, which often has a larger volume than the overland flow, had not been identified. As a result, "surface runoff" now has no meaning that is generally understood. It may include the interflow or exclude it; more often it includes from one-fourth to three-fourths of it. The writer believes that the unitgraph should represent "direct runoff", which is overland flow plus interflow. After making due allowance for interception and other direct losses, the difference between the volume of rainfall and that of direct runoff is the retention, while that between rainfall and overland runoff is the infiltration. This distinction is too often overlooked when applying the unitgraph.

Assumption (b) would restrict the name to unitgraphs that have been completely processed for application, leaving nameless those being processed or on file in a unitgraph bank. If the volume is to be a unit amount, it is important to admit non-standard units and dimensionless units. Actually, the prefix "unit" refers to unit time, not volume. In a letter to the writer in 1947, Sherman discussed the confusion of terms and stated his preferred definitions:

> "The unit hydrograph is a hydrograph of runoff from a given drainage basin, due solely to the volume of net rainfall (precipitation excess) occurring in a specified unit of time."
>
> "The unit hydrograph is a sequence of figures that express the typical distribution of runoff from a given basin."

The two definitions obviously apply to the observed and the processed unitgraph, respectively, without restricting the volume of either one. Sherman has also presented a drawing of a group of unitgraphs of different volume, applicable to the same river point.(4)

Assumption (c) was stated as a part of the original theory, but it is in no way essential. It is well established that a large flow will take longer to recede than a smaller one. It is fundamental, also, that of two unitgraphs for the same river point, if one has twice the volume of the other, it will recede twice as fast. Yet, it cannot recede from 100 cfs to zero in the same number of hours that the other recedes from 50 cfs to zero, if both unitgraphs follow the same normal depletion curve below 50 cfs. It is evident from the above that the normal recession of direct runoff has to become exponential as soon as the overland portion of the flow has diminished to a negligible amount.(5) The recession, then, is asymptotic to the time axis and would not reach zero unless the effects of evaporation, bank transpiration, and other natural losses were present. This concept brings the unitgraph and the depletion curve methods into complete accord and there is ample evidence that it is valid for practical purposes. It must be concluded that "length of base" is a meaningless term and that the unitgraph is as long or as short as the hydrologist wishes to make it.

Assumption (d), of course, provides a simple means of determining an apparent volume of the unitgraph. A fourth or more of the actual volume is often discarded by pinching off the recession, and when the graph is readjusted to unit volume, the peak becomes proportionally too high. The curtailment is entirely unnecessary because it is actually easier and better to apply the unitgraph in endless form.(6) This is especially true in continuous flow forecasting. In any case, if the recession is to be cut off, it should not be done until after the volume has been determined. The exponential portion of the recession can be expressed by the equation

$$Q = Q_0 k^t \tag{1}$$

where Q_0 is the discharge at a given instant, Q is the discharge after an interval of t time units, and k is the recession factor for one time unit. If Q is expressed in cfs and t in hours, the residual volume V_f at the given instant, in cfs-hours, is(2)

$$V_f = -\frac{Q}{\log_e k} = -\frac{0.434\,Q}{\log_{10} k} \tag{2}$$

It has already been shown that Eq. (1) is a correlary to the basic principle of the unitgraph. Table 1 gives values of the residual volume when Q is one cfs and k is the 24-hour recession factor, and is useful in finding the volume of an endless unitgraph.

Assumption (e) seems to have been accepted widely without critical examination. The flood hydrograph, of course, represents outflow from channel storage and the volume of storage diminishes during the recession. However, the rate of interflow in a unitgraph normally increases for some time after the point of inflection. The effect of channel storage upon a single unitgraph, after the flows of a series of them have become merged, is purely a matter of abstract reasoning. There is no doubt, however, that in small basins, the point of inflection of the flood hydrograph commonly occurs before all of the direct runoff has entered the stream channels.

Assumption (f) is the basis of all trial-and-error methods. A fair reproduction of a flashy hydrograph can substantiate a unitgraph, especially if there were two or more observed peaks or if the time of the rain bursts is known. A perfect reproduction of the direct-runoff hydrograph, starting at

TABLE 1. VALUES OF THE STORAGE FACTOR Z.

Volume of an Exponential Recession in cfs-hours per 1 cfs of initial discharge, where k is the 24-hour recession factor.

At the instant when the discharge is Q cfs, the remaining volume is QZ cfs-hours.

k	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0		5.21	6.13	6.84	7.46	8.01	8.53	9.03	9.50	9.97
.1	10.4	10.9	11.3	11.8	12.2	12.7	13.1	13.6	14.0	14.5
.2	14.9	15.4	15.8	16.3	16.8	17.3	17.8	18.3	18.8	19.4
.3	19.9	20.5	21.1	21.7	22.3	22.9	23.5	24.1	24.8	25.5
.4	26.2	26.9	27.7	28.4	29.2	30.1	30.9	31.8	32.7	33.6
.5	34.6	35.6	36.7	37.8	39.0	40.2	41.4	42.7	44.1	45.5
.6	47.0	48.6	50.2	51.9	53.8	55.7	57.8	59.9	62.2	64.7
.7	67.3	70.1	73.1	76.3	79.7	83.4	87.4	91.8	96.6	101.8
.8	107.6	113.9	120.9	128.8	137.7	147.7	159.1	172.3	187.7	205.9

zero flow and ending at a negligible discharge, would establish the unitgraph beyond any doubt. A reasonably good fit over most of the hydrograph, however, can often be misleading. Of the eight unitgraphs of Table 3, four will give perfect reproductions of the hydrograph of Table 2 as far as the end of its high-water portion. An even greater variety would have given "satisfactory" reproductions.

Assumption (g) must be accepted for certain purposes, such as the analysis of compound hydrographs. The increase of lag with stage in large, mature river systems is generally recognized. Similar effects in a small basin could easily be obscured because of the relatively short lag and the more prominent effect of the areal pattern of rainfall. There are likely to be relatively large errors in the lag of small-basin unitgraphs, resulting from the practice of steepening the recessions in order to shorten them.

The danger in assuming that a basin has a fixed lag time was made evident to the writer in the course of a routine study. Karnafuli River in East Pakistan drains an area of 3,800 sq. mi., and is subject to heavy monsoon rains lasting three to five days. The basin is quite rugged, being covered by pressure folds that appear as a series of high parallel ridges. Previous to the folding, the main stem of the river had been established on a flat alluvial plain. It is cut squarely through the ridges and is fed by tributaries that follow the synclines. The slopes of the major channels are extremely flat. All of the flood hydrographs are highly compounded and a painstaking analysis of them had to be made. It was found that the unitgraph lag, from the center of the unit rainfall duration to the hour when half of the runoff had occurred, varied from about 45 hours to more than 100 hours. The plotting of the values

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TABLE 2. COMPUTATION OF UNITGRAPH BY PROGRESSIVE ADDITION.

First trial sequence: 5, 3, 3, 3.

Time in days	Direct discharge, thousand cfs	Slide column	Unitgraph Reversed run	Unitgraph Forward run
		3		
		3		
0	0	3		(a)0
1	8.6	5		1.72
2	46.6			8.29
3	105.0		(a)-6.23	14.99
4	126.9		6.42	10.38
5	120.6		14.29	3.92
6	105.5		12.31	3.53
7	96.1		5.57	8.52
8	87.6		4.82	7.94
9	77.5		7.48	3.51
10	59.3		8.50	-.12
11	36.7		5.04	
12	* 23.6	3	* 2.89	
13	* 16.3	3	* 2.00	
14	* 11.3	3	* 1.38	
15	* 7.77	5	* .95	

* On exponential part of recession: k = 0.69.

(a) Corresponds to time of beginning (zero point) of adjusted unitgraph.

TABLE 3. UNITGRAPH BY PROGRESSIVE ADDITION.

Comparison of four successive trials.

Time in hours	Trial 1. 5, 3, 3, 3. Fd.	Rev.	Trial 2. 5, 3, 4, 3. Fd.	Rev.	Trial 3. 5, 4, 4, 3. Fd.	Rev.	Trial 4. 50, 45, 40, 30. Fd.	Rev.
0	0	-6.23	0	-4.19	0	-1.64	0	-.004
24	1.72	6.42	1.72	7.20	1.72	3.85	.172	.254
48	8.29	14.29	8.29	10.26	7.94	9.23	.777	.825
72	14.99	12.31	14.63	11.60	13.27	11.52	1.263	1.157
96	10.38	5.57	8.94	5.89	7.38	6.47	.676	.684
120	3.92	4.82	2.08	5.15	2.84	4.19	.327	.376
144	3.53	7.48	1.92	6.34	4.96	5.66	.517	.530
168	8.52	8.50	11.04	7.76	8.55	7.46	.790	.734
192	7.94	5.04	8.11	4.70	5.01	4.51	.431	.442
216	3.51	*2.89	.45	*2.67	1.68	*2.53	.220	*.247
240	-.12	*2.00	-1.52	*1.84	1.38	*1.75	.169	*.170
264		*1.38		*1.28	1.89	*1.21	.147	*.118
288		* .95		* .88	1.10	* .83	.072	*.081

* On exponential part of recession: k = 0.69.

left no doubt that the lag was closely related to the average stage of the flood event.

Assumption (h) represents the basic principle of the unitgraph method. Nevertheless, certain natural phenomena often alter the shape of the unitgraph. The areal pattern of the rainfall, of course, affects it, and particularl the relative distance from the gaging point to the center of rainfall. The steepness of the unitgraph is affected by the relative proportion of overland flow, which in turn often depends on the intensity and duration characteristics of the rain. A protracted rain at a rate roughly equivalent to the infiltration capacity of the ground can produce a large volume of interflow with little or no overland flow. In some instances, as was found in the Karnafuli study, the stage of the river may greatly affect the lag of the unitgraphs within a flood. Unitgraphs of ordinary shape are flattened proportionally as their lag increas es; this may or may not be true of unitgraphs having unusual features, such as multiple peaks. Finally, the method itself creates the picture of a series of unitgraphs, each of which preserves its identity as it accompanies the floo

The hydrologist must not forget that the laws of hydraulics govern primarily the total flow in the channel, rather than that of imaginary parcels of water within it.

Separation of Flows

One of the earliest methods of estimating the ground-water flow was to connect the low points of the discharge hydrograph by straight lines or smooth curves. This was later varied by the introduction of a rise and fall of ground-water flow coincident with that of each event of direct runoff. In the portion of a flood event that is being analyzed for unitgraph purposes, the writer has usually found no appreciable increase in ground-water flow unless it resulted from percolation previous to the storm that produced the flood event. The effect will appear sooner, of course, when the water table is higher. Presumably with bank storage in mind, hydrologists have often placed the ground-water crest much earlier than the bulk of the percolating water could possibly reach ground-water level, and the fall has been drawn much steeper than the rate of normal recession, making the peak erroneously high. At the same time, in assuming that the low points of the hydrograph represent ground-water flow, they have overlooked the residual of direct runoff that is likely to be fairly large at those points.

The writer is convinced that the observed normal recession characteristics of the stream are the most practical guide for separating the elements of flow, and that the minor effects of quick percolation in the bottom lands and changing head of ground water at the stream banks should be treated as direct-runoff phenomena. The normal recessions of ground-water flow and interflow each assume the form of Eq. (1), which plots as a straight line on semi-logarithmic paper.[5] This fact greatly simplifies the separation of ground-water flow. In the annual regimen of most streams, there is a season when a prolonged recharge of ground water occurs. For the rest of the year the semi-log hydrograph of ground-water flow consists mainly of straight lines at the slope of the normal ground-water recession.[2] Fairly heavy rains in the dry season often contribute no appreciable recharge, even though there may be considerable direct runoff. When a late-season recharge occurs, the rise may appear weeks after the rain that caused it, and is usually quite abrupt, the ground-water hydrograph resuming its normal slope almost immediately at the new level. If unusually hot weather occurs while the ground-water flow is low, evaporation and bank transpiration may cause an apparent decline that is much steeper than the normal recession. Usually, however, some part of the late-season hydrograph will clearly indicate the normal slope.

After a trial hydrograph of ground-water flow has been drawn, the direct discharge should be computed and plotted on semi-log paper with an expanded time scale. The shape of this hydrograph can be compared to the profile of a ripsaw, the rises being steep and the recessions tending to straighten. A characteristic slope of the straight portions will be seen. The hydrographs of ground water and direct runoff can then be mutually adjusted to accentuate the parallel appearance of the recessions without departing from a consistent behavior of the ground water. The recession factor for interflow is determined from the slope of the straight portions of the recessions of direct discharge.

The procedure used by the writer is as follows:

1. On semi-log calendar paper plot as many seasons of daily discharge as practicable, about 15 days to the inch. Include the years of unusually high discharge and at least one dry year, if possible.
2. Draw the trial hydrograph of ground-water flow as previously described. (The 24-hour recession factor is usually quite close to 0.98 and practically always greater than 0.90.)
3. Compute the trial direct discharge and plot it on semi-log paper, perhaps five or six days to the inch, or a scale that will slope the recessions between 30 and 60 degrees.
4. Examine the direct-runoff hydrograph and adjust it together with the ground-water hydrograph until both are plausible. This is often merely a matter of shifting the dates of recharge. Unless the ground-water flow is a large part of the total, minor adjustments are justified only where they are needed to confirm the normal recession factor for interflow.
5. Determine the characteristic slope of the interflow recession and draw a line with that slope. Find the 24-hour recession factor, which is the ratio of the ordinates at the end and the beginning of one day. Its value is nearly 0.80 in some large basins and is often less than 0.50 in extremely small ones. It is usually more convenient to pick off the factor for an extended period and reduce it logarithmically to its 24-hour equivalent.
6. Select the events that are to be analyzed for unitgraphs. Obtain, if possible, instantaneous discharge values for those events at intervals that will define the graph in detail, especially the peaks, troughs, and the hour when the first rise began. If only the daily mean values are available, the actual hydrograph, should be estimated from them. Daily values obtained from single readings should be treated as instantaneous discharge. Rainfall records often help to determine the hour of the first rise.
7. From the observed discharge, subtract the ground-water flow and plot the hydrograph of direct discharge on semi-log paper. Draw the recessions of residual flow at the beginning of the event and at the end, as straight lines having the normal recession slope (Fig. 1).
8. At the beginning of the event, subtract the residual flow of the previous event from the direct discharge up to the time when the residual becomes negligible. Plot the new net values on the same sheet. The hydrograph of the direct-runoff event is now ready for analysis (Fig. 2)
9. Some unitgraph procedures require that the volume of direct runoff be determined from the hydrograph. When this is desired, determine the volume from the beginning of the event to an early point on the exponential recession by the usual methods. Using the storage factor from Table 1 (or by Eq. (2)), determine the remaining volume under the recession.

The Principle of Progressive Addition

The simplest way to obtain a unitgraph is to find and utilize a suitable isolated unit event. Such events are rare; usually it is necessary to separate a material amount of overlapping runoff. The uncertainties of that process can

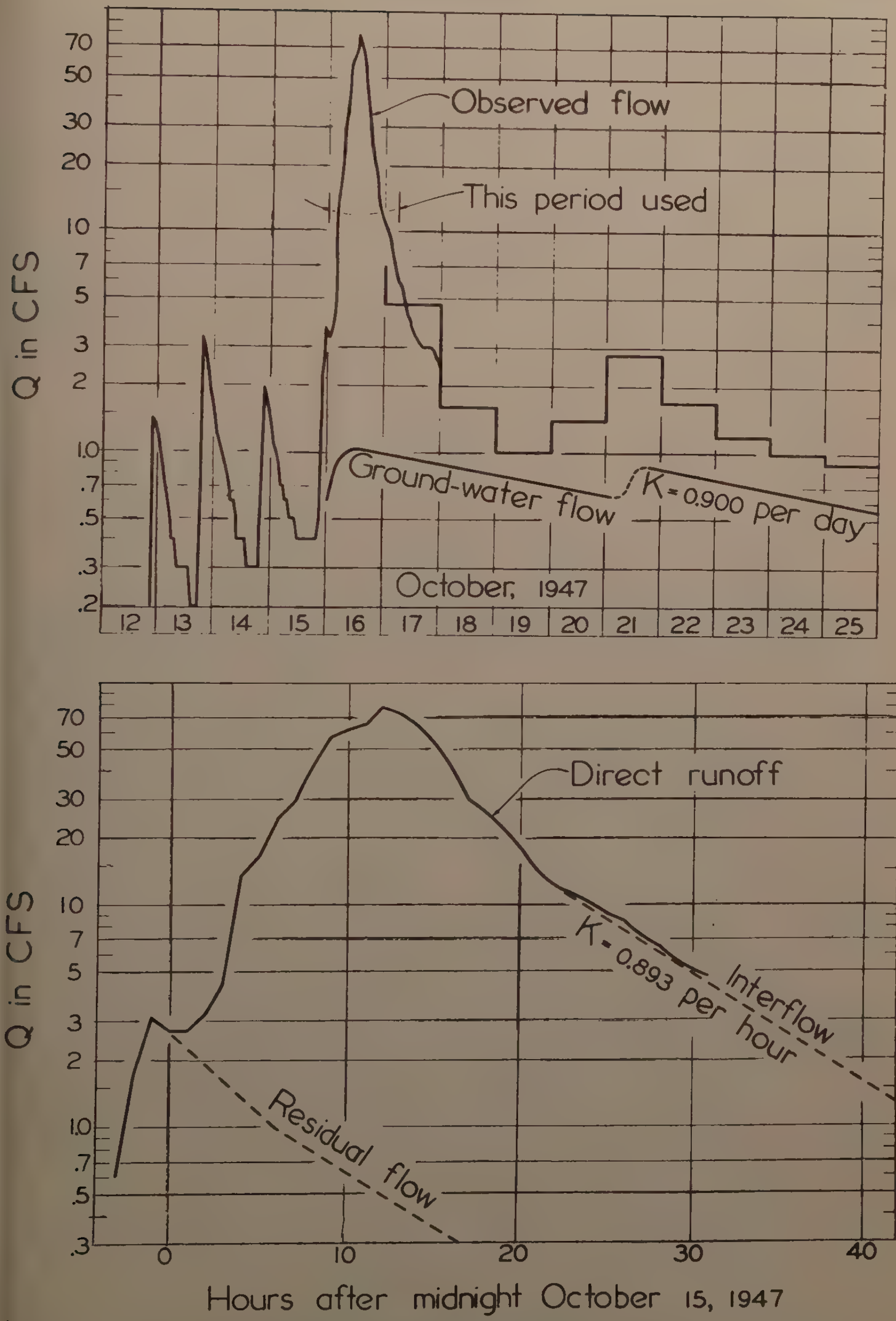

Fig. 1. Separation of the base flows, Castle Creek at Central Sierra Snow Laboratory.

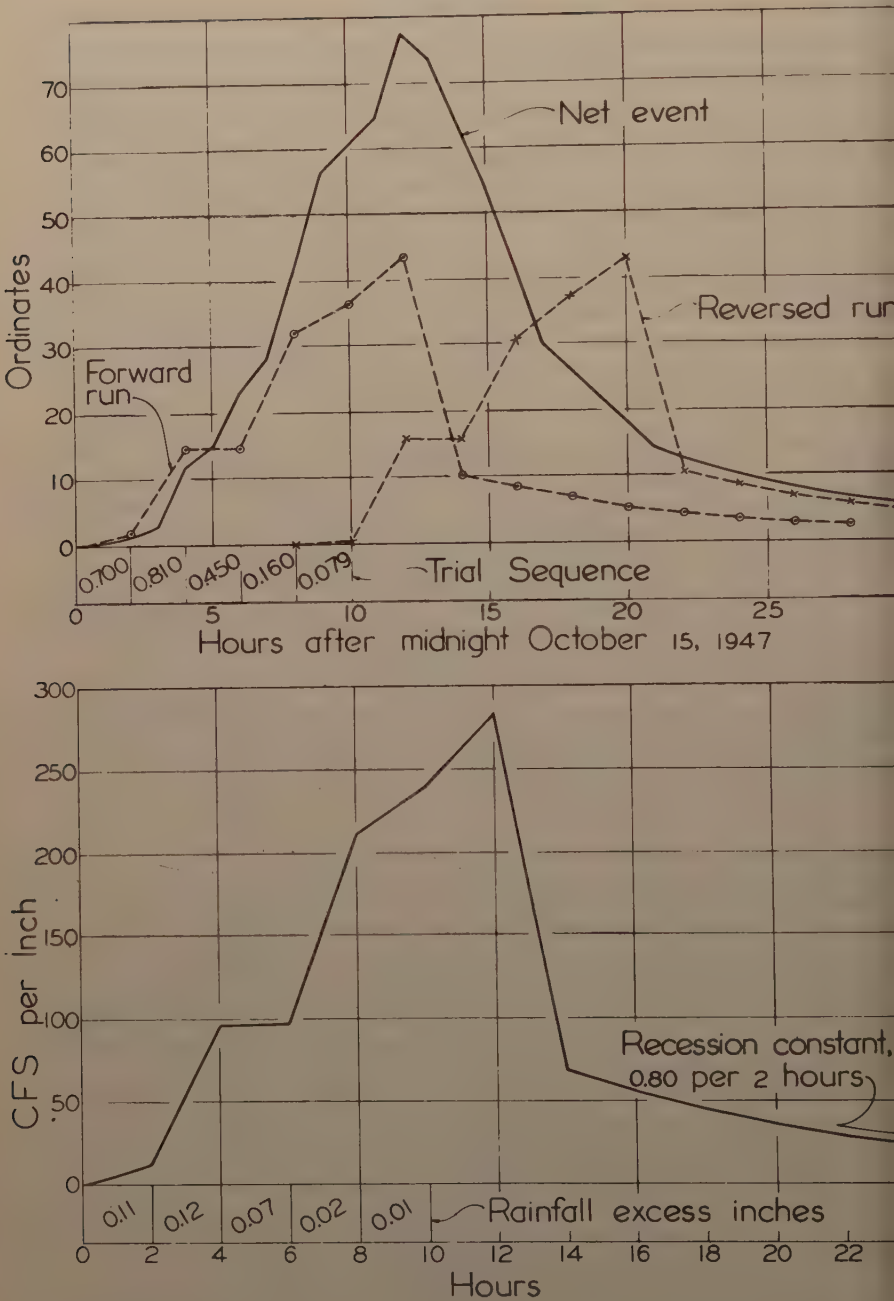

Fig. 2. Two-hour unitgraph, Castle Creek.

be minimized by the use of the semi-log hydrograph. Often, however, the most suitable events are highly compounded. There is always the possibility, also, that unitgraphs from simple events may not be characteristic of more protracted floods in the stream. A simple and positive method of deriving a unitgraph from a compound event, with or without rainfall records, is therefore of great potential value.

The most reliable breakdowns of compound hydrographs are those performed by analytical methods. Assume that an event of direct runoff results from the four increments of excess rainfall R_1, R_2, R_3, and R_4 inches, occurring in successive units of time. The ordinates of the hydrograph, at unit intervals, are Q_1, Q_2, Q_3, etc., and those of the unitgraph are U_1, U_2, U_3, etc. By the unitgraph theory, the ordinates of the hydrograph can then be expressed

$$
\begin{aligned}
Q_1 &= U_1R_1 \\
Q_2 &= U_2R_1 + U_1R_2 \\
Q_3 &= U_3R_1 + U_2R_2 + U_1R_3 \\
Q_4 &= U_4R_1 + U_3R_2 + U_2R_3 + U_1R_4 \\
Q_5 &= U_5R_1 + U_4R_2 + U_3R_3 + U_2R_4
\end{aligned}
\tag{3}
$$

and so on for the rest of the ordinates. If the R-values could be known exactly, Eqs. (3) could be solved as normal equations to determine the ordinates of the unitgraph. A variation of the method is to fit the equations to the observed hydrograph by multiple regression. Procedures of these types are discussed in references (7, 8 and 9).

A different approach based on Eqs. (3) has been in limited use for several years.[6] Its origin is not known to the writer and since the method is tedious and sometimes deceptive, it has never become popular. It is necessary at this point to explain the process, which may be called "progressive addition". Trial values of R_1, R_2, R_3, to R_n are first set up, having their sum equivalent to the volume of the compound event. U_1 is determined from Q_1 and R_1. U_2 is obtained by subtracting U_1R_2 from Q_2 and dividing the difference by R_1, and so on. The ordinates are plotted as fast as they are determined and a smooth mean curve is maintained between the points. By inspection of the curve, adjustments of the R-values are made and the U-values are revised accordingly. This method gives a leverage to small errors that greatly magnifies their effects, and the process usually breaks down in wild values as soon as the peak is passed. It is then necessary to go to the far end of the compound event and work backward toward the peak.

The basic equations of the method of progressive addition are obtained from Eqs. (3) and can be written as follows:

$$
\begin{aligned}
U_1 &= \frac{Q_1}{R_1} \\
U_2 &= \frac{Q_2 - U_1R_2}{R_1} \\
U_3 &= \frac{Q_3 - (U_2R_2 + U_1R_3)}{R_1}
\end{aligned}
\tag{4}
$$

$$U_4 = \frac{Q_4 - (U_3R_2 + U_2R_3 + U_1R_4)}{R_1}$$

The series of equations can be extended to U_n, following the pattern above. Only as many R-terms appear as there are increments of excess rain, all others becoming zero. In the application, the UR multiplications are performed with a calculating machine and the products are allowed to accumulate on the dial. The actual computations are quite rapid and are greatly facilitated by first entering the Q-values in order vertically on the work sheet, leaving an adjacent column for the U-values as they are determined. The R-values are then written in a column, in inverted order, on a separate strip of paper that is used as a slide.

The unitgraph derived on the forward run will commence with the first increment of excess rain, while zero time of the other unitgraph will be at the start of the last increment (Fig. 2). The two are therefore out of phase by a period equal to the total duration of excess rain less one time increment. In order to compare them, the unitgraph of the reversed run must be shifted forward that number of hours.

The procedure described above has proved quite valuable in special problems but was never entirely practical for routine use. It was always difficult to obtain a satisfactory match of the two runs as they passed the crest. When a trial rainfall increment was changed, the sequence had to be readjusted to hold its volume constant. The reversed run had to be started at a point of negligible flow and carried back through a long recession. The shape of the derived unitgraph was affected by the operator's judgment and there was no inherent measure of its accuracy. The adjustments obscured the natural irregularities of the graph, and a zone between two peaks usually could not be defined at all.

The writer has found that the shortcomings of the procedure are eliminated, and its operation made simple and rapid, by three significant changes:

1. To start the reversed run at the first regular ordinate in the exponential part of the recession and extend the run back to zero time. The forward run is made the same length as the reversed run.
2. To make no adjustments during any pair of runs. Between runs, only the excess rainfall sequence is adjusted.
3. To express the excess rainfall neither in inches nor millimeters, but merely as a series of numbers, paying no attention to their sum. Any single number may then be altered at will. The unitgraph will have its ordinates in non-standard units but will be no less valid for that reason.

In this revised procedure, the objective is a fair agreement of the two unadjusted runs, forward and reversed, which is accomplished solely by altering the trial rainfall numbers. The method is rapid, self-checking, and almost foolproof. Skill developed in practice can greatly speed the operation but will not affect the results. The revised procedure will be described in detail in ensuing sections of this paper.

The Reversed Run

Until an approximate sequence of R-values has been established, it is usually simpler to start with the reversed run, rather than the forward run.

In order to start the run at any discharge rate except zero, a short series of U-values must first be set up at the starting point, so that the initial application of the rainfall sequence can be made. Consider a compound hydrograph of direct runoff, extending into a period of normal recession. Assume that six hours has been selected as the appropriate unit time, that the trial increments of excess rain are expressed by the four numbers R_1, R_2, R_3, and R_4, in that order, and that the normal recession slope is expressed by the 6-hour recession factor k. Let Q_d be the first regular ordinate after the recession becomes exponential, with Q_c, Q_b and Q_a following at 6-hour intervals. Each of the latter three values, then, is equal to k times the value preceding it. The same will be true of the corresponding ordinates of the unitgraph and the latter may readily be computed from their Q-terms by applying as a factor the ratio of Q_d to the sum of the four products R_1Q_a, R_2Q_b, R_3Q_c, and R_4Q_d.

The operations in setting up and performing the reversed run with four rainfall increments are as follows:

1. The trial R-values are assumed to have been entered on the slide in inverted chronological order, starting with R_1 at the bottom. The entire series of Q-values at unit intervals is assumed to have been entered in a column of the work sheet in chronological order starting at the top and ending with Q_a at the bottom.
2. Place the slide so that R_1 is opposite Q_a. Multiply R_1Q_a, R_2Q_b, R_3Q_c and R_4Q_d, letting the four products accumulate on the machine. Divide Q_d by the sum. Apply the quotient as a factor to Q_a, Q_b, Q_c and Q_d in turn, to obtain U_a, U_b, U_c and U_d, in that order. Enter the four U-values in a column of the work sheet, opposite the Q-values with the same subscripts.
3. Without moving the slide, check the results by accumulating the products R_1U_a, R_2U_b, R_3U_c and R_4U_d. Their sum should equal Q_d.
4. The actual run is started by raising the slide one line. Accumulate the bottom three RU products and subtract from the Q-value opposite R_4. Divide the difference by R_4 and enter the quotient in the U column, opposite R_4. Raise the slide and repeat, continuing in this manner until the run is finished. A numerical example of the process is given in the following section.

Deriving the Unitgraph

The general procedure of deriving a unitgraph from a compound hydrograph, using the revised method of progressive addition, is as follows:

1. Separate the direct runoff by the method previously outlined.
2. Estimate the approximate duration of excess rainfall or choose a duration at random, if necessary. Select a time interval to be used as the unit duration. This should not exceed about one-third of the probable lag time, as measured to the midpoint of the volume of the unitgraph.
3. Set up a trial series of R-numbers to represent the excess rainfall increments. Their sum is of no importance; only their ratio is significant. For the first trial use small whole numbers. If there are no rainfall data, use such a sequence as 5, 5, 5, 5. If more than five increments are assumed, the unit duration should be doubled for the earlier runs in order to reduce the number of terms in the sequence. A

subsequent trial sequence can be expanded to splitting each of the numbers.

4. Make the reversed run. If a negative U-value occurs, stop the run. Unless the run was almost complete, change one or more R-values on the slide and try again.
5. If the run was complete or nearly so, shift the time earlier by the amount of the phase correction (the length of the rainfall sequence less one time increment) and plot the U-values as a hydrograph. A small scale should be used. It is best to divide up the cross-section sheet and use one part for each trial sequence, identified by its R-numbers.
6. When a reversed run has been completed, or nearly so, without negative U-values, make the forward run without changing the numbers on the slide. The run is started with the slide at the top of the column. The first U-value is entered as zero, opposite the zero Q-value. Set the slide with R_1 opposite the next Q-value and divide the latter by R_1. Enter the quotient as the second U-value. Lower the slide one line. Multiply R_2 by its U-value and subtract from Q opposite R_1. Divide by R_1 and enter as the third U-value. Lower the slide one line. Accumulate the U-products of R_2 and R_3 and subtract from Q opposite R_1. Divide by R_1 as before. Lower the slide and continue.
7. Stop the run if negative values appear, otherwise continue it through as many unit periods as the reversed run. In either case, plot the two runs together for comparison. No phase correction is applied to the forward run.
8. Change one of the R-numbers, selected after a study of the previous plotting, and make a new pair of runs. Plot the runs and study the effect of the change; whether it improved the fit, where, and how much. It should now be possible to judge which R-number can be changed most advantageously for the next run, and in which direction. It may appear that not enough increments, or too many, are being used. However, a rough conformity of the two runs will become evident before any increments need to be added or discarded.

Stated briefly, the initial objective is to select a sequence of R-numbers that will nearly complete a pair of runs with no negative ordinates. The final objective is to bring the two runs into fair agreement. If rainfall records are available, the first trial should achieve the initial objective. The skill needed to speed the rest of the process is quickly acquired. Note that one of the R-numbers can be kept unchanged if desired. Since the first and last terms serve as divisors, it is helpful to keep them in round numbers as long as practicable, fractional adjustments being made first to the other terms.

Numerical Example

An example of the first trial is shown in Table 2. The flood event occurred on Karnafuli River at Rangamati, East Pakistan, in July 1947. Unit time has been taken as one day. Because hourly discharge values were available, the days do not start at midnight, and zero time is the hour of the start of the flood. The Q-values represent momentary direct discharge in thousands of cfs, at 24-hour intervals. From available rainfall records, four days of excess rainfall were assumed, in the ratio 5, 3, 3, 3; one trial perhaps being avoided by starting with those numbers instead of 5, 5, 5, 5. The starting position of the slide, for the reversed run, is shown at the bottom of the "slide"

column in Table 2 and the starting position for the forward run is shown at the top. The starred values of the discharge column are on the exponential part of the recession (k = 0.69 per day), and those starred in the reversed run are proportional to them. The reversed run was performed as follows:

1. Place the slide opposite the starred numbers of the discharge column. Use those values in place of unitgraph ordinates to compute a fictitious discharge at 12 days. This will be (5 x 7.77) + (3 x 11.30) + (3 x 16.30) + (3 x 23.60). The products are allowed to accumulate on the dial and their sum is 192.45. Divide the actual discharge (23.6) by the latter to obtain the factor 0.1226. Apply that factor to the starred values of the discharge column to obtain the corresponding unitgraph ordinates. Enter the latter on the work sheet.
2. Clear the machine. Without moving the slide, test the four unitgraph ordinates by computing the discharge at 12 days. This will be (5 x 0.95) + (3 x 1.38) + (3 x 2.00) + (3 x 2.89), accumulating 23.56 on the dial, which agrees with the value 23.6 in the discharge column.
3. Clear the machine. Move the slide up one line on the work sheet. Accumulate the three products 5 x 1.38, 3 x 2.00, and 3 x 2.89. Subtract the total (21.57) from the discharge opposite the top figure of the slide and divide the difference (15.13) by the top figure. Write the result (5.04) as the unitgraph ordinate opposite the top figure of the slide. Clear the machine.
4. Move the slide up a line at a time and continue the computations in the same manner. End the run at 3 days, which corresponds to zero time on the forward run (in this instance the negative ordinate at 3 days would have ended the run automatically).

The forward run was performed as follows:

1. Place the slide so that its bottom figure is opposite the top figure of the discharge column. Divide 8.6 by 5 and enter 1.72 in the unitgraph column.
2. Move the slide down one line. Multiply 1.72 by 3 and subtract 5.16 from the discharge opposite the bottom figure of the slide. Divide the difference by 5 and enter 8.29 as the unitgraph ordinate.
3. Move the slide down one line. Accumulate (3 x 1.72) and (3 x 8.29) and subtract the sum from 105.0. Divide by 5 and enter 14.99 as the unitgraph ordinate.
4. Move the slide down a line at a time and continue in the same manner. Stop on the fourth line from the bottom, or in any case, when a negative ordinate appears.

Since there were four numbers in the trial sequence, the reversed run must be shifted three lines upward for direct comparison and plotting with the forward run. Table 3 shows the results of the first four trials. If the unitgraph were to be used in a research study, two or three more trials might have been desirable. For example, the sequence 50, 44, 39, 28 will bring the two runs much closer together. The average of those runs, however, will give a unitgraph that hardly differs proportionally from that of the fourth trial. In fact, the average of the third trial would be a satisfactory unitgraph for all ordinary purposes. Fig. 3 shows the plotted pairs of runs made with five trial sequences.

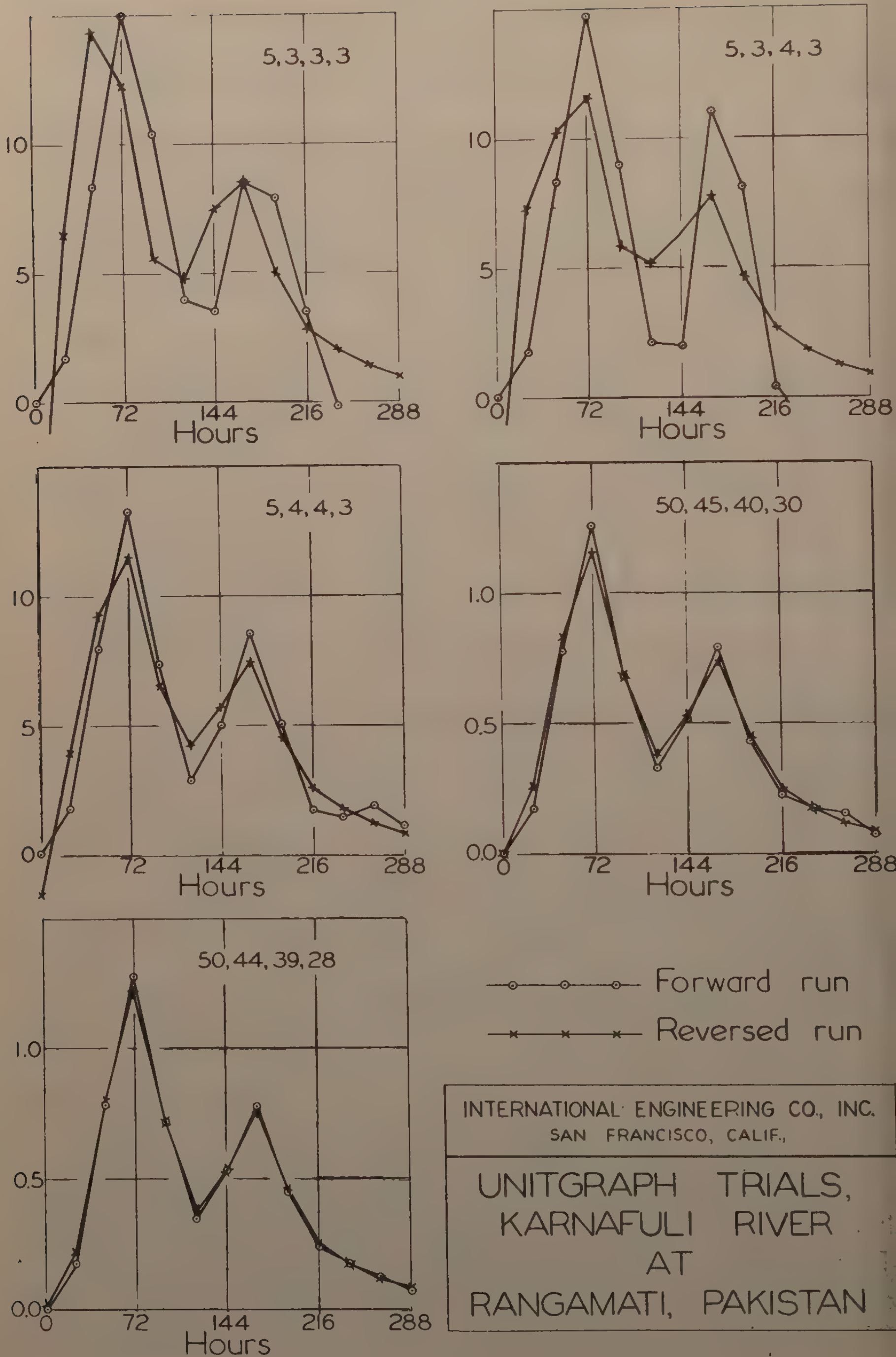
5, 3, 3, 3
5, 3, 4, 3
5, 4, 4, 3
50, 45, 40, 30
50, 44, 39, 28
Hours
Forward run
Reversed run
INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.,
UNITGRAPH TRIALS, KARNAFULI RIVER AT RANGAMATI, PAKISTAN

Fig. 3

It is not necessary to know the volume of the event of direct runoff unless a retention study is being made. If it is desired, however, the volume can be determined as described earlier, or by multiplying the numerical volume of the adopted unitgraph by the sum of the numbers in the trial sequence that produced it. In the present case, the volume of the event is 946 thousand cfs-days, which is equivalent to 9.25 inches of excess rainfall. The daily increments corresponding to the trial sequence 50, 44, 39, 28 are 2.87, 2.53, 2.24 and 1.61 inches, respectively, while those corresponding to the fourth trial are 2.80, 2.53, 2.24 and 1.68 inches, and those of the third trial, 2.89, 2.31, 2.31 and 1.74 inches.

As the rainfall sequence is developed, it has a tendency, no doubt, to compensate certain inconsistencies of the basic data. Note, however, that in this revised procedure each ordinate of the unitgraph is derived twice, from entirely different parts of the record. For instance, the second peak was derived, on the forward run, from the first seven days, while on the reversed run it was derived from the eleventh through the seventeenth day. Therefore, if the fit of the two runs is good, their average unitgraph can safely be accepted for that flood event. It is not unusual to find events for which the runs cannot be brought into acceptable agreement, because of variation in the rainfall pattern or errors in the discharge record.

Lag of a Unitgraph

Lag as originally defined by Horner and Flynt,(10) that is, measured between the centers of mass of the excess rainfall and of the runoff, is the best index of the "difference in phase between salient features of the rainfall and runoff rate curves" because it is not affected by the duration of excess rainfall. There are three other standards of lag in common use. One of these is measured between the midpoint of the unit rainfall period and the peak of the unitgraph. Another is measured from the beginning of the unit period to the instant when the ordinate of the summation hydrograph(11) reaches 50 per cent, or half of the rate of excess rainfall. The third is measured from the midpoint of the unit period to the instant when half of the unitgraph volume has passed the gage. These three standards involve less labor than the one of Horner and Flynt, but the values obtained with them are influenced by the unit rainfall duration. It can be demonstrated that there is no difference, numerically, between the lag by the latter two definitions. The last is the one used by the writer in the Karnafuli study.

Several equations for relating lag to the physical characteristics of the drainage basin have been proposed. The square root of the slope (probably borrowed from the Chezy formula) appears in most of them. The writer has not seen a successful correlation of lag with slope in basins having comparable topography. Perhaps there have been enough random errors to obscure the relationship. More likely, the relation between the slope and the cross-section of a naturally-developed channel tends to nullify the effect of slope upon lag. It is true, also, that in steep headwater channels nearly all of the energy of the fall is likely to be dissipated by cascades and rollers, so that the actual progress of the water may be relatively slow.

The effects of river stage and rainfall pattern upon lag have already been discussed. It is evident that more study of the entire subject of lag relations is badly needed. Unitgraphs used for that purpose should be so derived that

they will not be biased by the individual tendencies of the investigators. The relationships of basin characteristics to lag can, no doubt, be detected more readily if the Horner and Flynt definition is used for establishing the lag, and if the unitgraphs are treated, for lag purposes, as endless. To determine the lag under these conditions, consider the unitgraph to be divided into two parts, the dividing line being at the earliest regular ordinate on the exponential recession. Let q represent the discharge at that instant. Let T_c be the elapsed time from the start of rise to the centroid of the entire unitgraph, M_i the time-volume moment of the initial part about zero time (start of rise), V_f the runoff volume of the final part, T_i the duration of the initial part, and V the total runoff volume of the unitgraph. Then(6)

$$T_c = \frac{M_i + V_f \left(T_i + \frac{V_f}{q}\right)}{V} \tag{5}$$

Half of the unit duration of excess rain must be subtracted from T_c to obtain the lag. For the unitgraph of Table 4 (obtained from the fourth trial of Table 3) the lag may be computed as follows:

Since the ordinates of the unitgraph are expressed in non-standard units, the discharge unit is designated "cxs". T_i is nine days and q is 0.234 cxs. M_i is found to be 22.87 day-cxs-days by summing the moments of the first nine days:

Mean Time	Volume	Moment
0.5	0.106	0.05
1.5	.507	0.76
2.5	1.006	2.52
3.5	.945	3.31
4.5	.516	2.32
5.5	.438	2.41
6.5	.643	4.17
7.5	.599	4.48
8.5	.335	2.85
	5.095	22.87

The volume, V_f, in cxs-hours remaining to run off after the ninth day is the product of q and the storage factor from Table 1. Since k is 0.69 per day, the storage factor is 64.7 and V_f is 15.1 cxs-hours or 0.631 cxs-days. The total volume, V, is equal to V_f plus 5.095, or 5.73 cxs-days. By Eq. (5), T_c becomes 5.28 days, or 127 hours. Subtracting 12 hours (half of the unit duration), the lag by the definition of Horner and Flynt is found to be 115 hours.

In the above example, the center of mass of the pluviagram was assumed to be at the midpoint of its duration. On that assumption, the lag of 115 hours is independent of the 24-hour unit duration used in the study. If a 12-hour unit

TABLE 4. COMPUTATION OF DIMENSIONLESS UNITGRAPH.

(From average of fourth trial, Table 3)

	Derived Unitgraph			Dimensionless Unitgraph	
T, hours	Q, cxs	Volume, cxs-days	Accum. volume	Abscissa .926 T	Ordinate 18.9 Q
0	0			0	0
24	.213	.106	.106	22	4.02
48	.801	.507	.613	44	15.1
72	1.210	1.006	1.619	67	22.8
96	.680	.945	2.564	89	12.8
120	.352	.516	3.080	111	6.63
144	.524	.438		133	9.87
168	.762	.643		156	14.4
192	.436	.599		178	8.22
216	* .234	.335	5.095	200	* 4.41
288	* .077			267	* 1.45

* Starred values are on exponential part of recession.

"cxs" represents a non-standard unit of discharge.

k (as derived) = 0.69.

From Table 1, Z = 64.7. Volume = $5.095 + \frac{.234\ Z}{24} = 5.73$ cxs days.

Lag + ½ duration = 108 hours (Lag = 96 hours).

Percent of $(L + \frac{1}{2} d) = \frac{100\ T}{108} = .926\ T$.

$$\frac{Q\ (L + \frac{1}{2}\ d)}{\text{Volume}} = \frac{108\ Q}{5.73} = 18.9\ Q.$$

duration had been used, T_c would have been 6 hours shorter and the computed lag would have been 115 hours as before. However, by the second and third of the substitute definitions, the lag of the 24-hour unitgraph would have been 96 hours and would have reflected the unit duration in some degree. If the lag had been measured to the peak, it would have been only 60 hours and would have been affected even more by the unit duration.

Processing of Unitgraphs

All of the values in Tables 2 and 3, except those of time, are in non-standard units. Before the unitgraph is applied, it must be adjusted so that its shape is consistent with the unit duration chosen for its application, its lag is correct for the basin, and its volume is equal to that of a standard unit depth of rainfall over the basin. If the lag and duration are not to be changed, the unitgraph can be prepared for application merely by adjusting its ordinates proportionally to give it the appropriate volume. For processing nearly all unitgraphs, however, the writer prefers to convert them to dimensionless form.(6) By this method, unitgraphs with different unit duration, from different basins, and with different discharge units, are reduced to a common basis for comparison and averaging. The adopted dimensionless unitgraph is then evaluated, in a single operation, for application with a given duration, lag, and basin area. The adjustment is based on two quantities:

1. The lag plus half of the unit duration of excess rain, in hours.
2. The volume of the unitgraph in "discharge-days"; that is, cfs-days if the ordinates are in cfs. The discharge need not be expressed in any standard units.

In converting the unitgraph into dimensionless form, its ordinates are multiplied by a factor equal to (1) above, divided by (2) above. Its abscissae are expressed in per cent of (1) above. In evaluating the dimensionless unitgraph for application the process is reversed. Quantity (1) then becomes the basin lag plus half of the adopted unit time. Quantity (2) becomes the volume equivalent of one inch (or one millimeter) of rain over the basin.

The unitgraph of Table 4 was converted into dimensionless form as follows The volume of the unitgraph is 5.73 cxs-days. The lag plus one-half duration was measured from the start of the unitgraph to the instant when half of the volume had been discharged, and was found to be 108 hours. The conversion factor for the ordinates is 108 divided by 5.73, and that for the abscissae is 100 divided by 108. In Fig. 4 this unitgraph is compared directly with that of a mountain brook having only a thousandth as much drainage area as the Karnafuli.

Dimensionless unitgraphs are best compared when plotted on semi-log paper. In averaging two or more graphs, the average ordinate and average abscissa of the peaks should first be determined, so that the process will not flatten the crest. In the lower parts, the abscissae can be averaged at successive heights. When a composite graph has been evaluated for application, its volume should be compared with that of a unit depth of rain over the basin. This will verify the accuracy of the averaging or indicate any final adjustment that may be necessary.

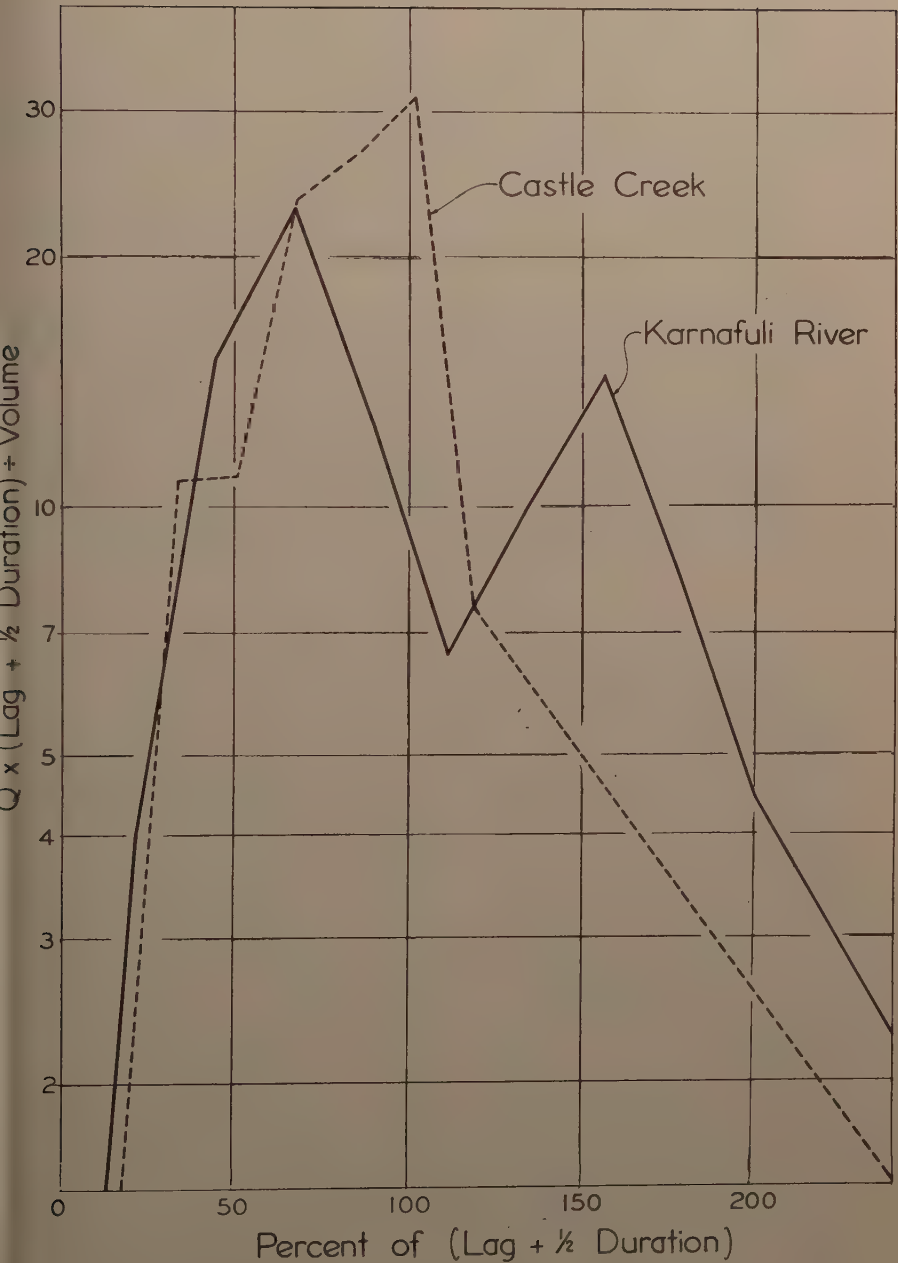

Fig. 4. Dimensionless unitgraphs.

CONCLUSION

Karnafuli River was chosen as an example because its hydrographs cannot be analyzed by conventional methods. The river is fed by protracted monsoon rains and its flow is sluggish. The flood events are so smoothly compounded that the common methods of deriving the unitgraph are useless. A solution by normal equations or by multiple correlation would have been defeated by the lack of detailed rainfall data or by the variable lag time. Any other established method might have failed to disclose the double crest of the unitgraph. It was interesting to discover that the characteristic second crest was present in all eight unitgraphs that were derived for Karnafuli River by the writer's procedure.

In the practice of flood hydrology there are no good substitutes for experience and judgment. The personal equation, however, should be removed from the actual technical processes as far as it is practicable to do so. It is generally recognized that basic streamflow data of good quality are likely to be much more consistent than the unitgraphs that are obtained from them. If the results of individual studies can be made more comparable, the unitgraph method can be more thoroughly explored. The methods outlined in this paper are submitted as proposed steps in that direction. The writer believes that they can readily be adapted for solution by the electronic digital computer, which would greatly facilitate the processing of large quantities of data.

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SPILLWAY DESIGN FOR PACIFIC NORTHWEST PROJECTS

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SYNOPSIS

This paper summarizes the design criteria and characteristics for spill-
vays in five Pacific Northwest Projects which include Chief Joseph, McNary, The Dalles, Detroit, and Cougar. The spillway crests for Chief Joseph and The Dalles are underdesigned to seventy-five per cent of the maximum heads. Model studies were used extensively in the design of four of the five spillways described in the paper.

INTRODUCTION

Hydraulic design criteria and characteristics of spillways for five Pacific Northwest Projects, McNary, Chief Joseph, The Dalles, Detroit, and Cougar, are discussed in this paper. These five projects were designed and constructed, or are being constructed, and are operated by the Corps of Engineers. All are multiple-purpose projects with maximum heads ranging from 0.5 ft at The Dalles to approximately 447 ft at Cougar.

During the period of design of these and other Pacific Northwest Projects, the Corps of Engineers has attempted to standardize the design of spillways.(1) Perhaps the most outstanding features in this regard have been the acceptance of (1) crests designed for 75 per cent of the maximum head, (2) semicircular pier noses, and (3) gate slots with 1 on 12 taper on the downstream side of the slots.

Model studies were used extensively in the design of four of the five spillways discussed in the paper; the spillway for Cougar was not model tested. Based on the model studies and other criteria, a number of revisions were

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made to the initial designs of the spillways, resulting in a considerable numb of improvements and monetary savings.

General Description of Projects

McNary

The McNary Project is located on the Columbia River between the states of Oregon and Washington 292 miles from the Pacific Ocean. Major features of the project are a 22-bay spillway, a 14-unit powerhouse, an 86- by 675-ft navigation lock, two fish ladders, fish locks, concrete nonoverflow sections, and rock-fill abutments. Maximum head on the dam is 92 ft.

Chief Joseph

The Chief Joseph Project is located on the Columbia River in the state of Washington approximately 546 miles from the mouth of the river. The proje consists of a 19-bay spillway, a 20-unit powerhouse (with provision for seve additional units), a concrete nonoverflow section between the spillway and powerhouse, and rock-fill abutments. Neither navigation nor fish passage fa cilities are provided at the Chief Joseph Project. Maximum head on the dam is approximately 169 ft.

The Dalles

The Dalles Project is located on the Columbia River 194 miles from its mouth. It is situated between the states of Oregon and Washington about 3 miles upstream from the city of The Dalles, Oregon. Salient features of the project are a 23-bay spillway, a 14-unit powerhouse (with provisions for eig additional units), an 86- by 675-ft navigation lock, two fish ladders, a fish lock, and a rock-fill closure dam at the left abutment. The maximum head o the dam is 90.5 ft.

Detroit

Detroit Dam is situated in the Willamette River Basin in the state of Oregon. The North Santiam River, on which Detroit Dam is located, is subject to low flows from July through October and relatively high flows dur ing the balance of the year. The project, located about 35 miles southeast o Salem, Oregon, is a concrete gravity-type structure with a six-bay spillway, four flood control conduits through the spillway section, and a two-unit powe house. The right spillway bay is equipped for studies on air entrainment in high-velocity flow. An 8-ft test conduit is installed in the left concrete abutment for tests on high-head control valves. The maximum head on the dam is approximately 375 ft.

Cougar

Cougar Dam, also in the Willamette River Basin, is located on the South Fork McKenzie River, 4.4 miles above the confluence with the McKenzie River, and about 42 miles east of Eugene, Oregon. The project will consist of a rock-fill dam with a 2-bay, gated spillway at the right abutment and an intake structure near the left abutment leading to a flood control outlet tunne and a penstock to a 2-unit powerhouse. Maximum head on the dam is approximately 447 ft.

McNary Spillway

McNary is a low-head type dam. The spillway consists of twenty-two 50-t-wide bays with 10-ft-wide piers. It is designed to pass 2,200,000 cfs with ı maximum head of 65.5 ft on the spillway crest. The maximum flood of record, 1,150,000 cfs in 1894 (adjusted in 1948 to 1,190,000 cfs), will be passed with normal pool, Elev. 340.

Initially the spillway was designed for 24 bays as determined from the discharge equation for broad-crested weirs:

$$Q = C\,(L - K_p n H)\,H^{3/2}$$

where Q = discharge in cfs

C = coefficient of discharge of crest without piers

L = net length of spillway crest exclusive of piers

K_p = pier contraction coefficient

n = number of pier contractions

H = total head (includes velocity of approach)

A value of $K_p = 0.040$ at maximum or design head of 65.5 ft on the crest was used to determine the required length of spillway, resulting in twenty-four 50-ft bays.

Hydraulic Model Studies

Studies in a 1:36 scale model of three bays of the initial design at the Bonneville Hydraulic Laboratory showed that the pier contraction coefficient was not a constant value, that it varied with the total head on the crest, and that for the airfoil-shaped pier nose it was a much smaller value than the 0.040 used originally. Results of the model tests revealed that the value of K_p became negative (-0.0033) at the design head of 65.5 ft which appreciably increased the discharge per bay beyond that computed in the initial design. By using the model values of K_p (Fig. 4), it was possible to revise the initial design in one of three ways:

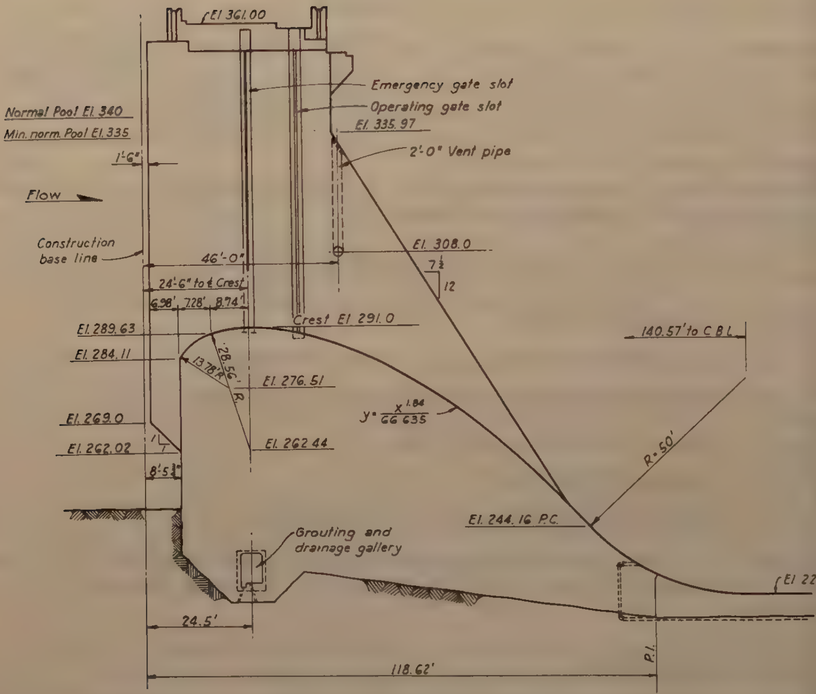
a. Reduce the number of spillway bays from 24 to 22.
b. Lower the maximum pool by 3.6 ft.
c. Raise the spillway crest by 3.75 ft.

It was found that reducing the number of spillway bays from 24 to 22 was the most advantageous and economical of the three alternative revisions.

A section through the spillway is shown in Fig. 1. The spillway crest shape was designed to fit the nappe profile for discharge at the design head of 65.5 ft (2,200,000 cfs at pool Elev. 356.5) in accordance with the Bureau of Reclamation data presented in "Studies of Crests for Overfall Dams".(2)

Crest Profile

The equation for the downstream portion of the crest profile is $x^{1.84} = 66.635y$. The upstream portion of the crest profile was approximated by compound curvature. The discharge capacity of the crest was determined in the 1:36-scale model with and without piers for heads that ranged from 25



McNARY SPILLWAY SECTION FIG. I

to 75 ft. For the design head of 65.5 ft, a value of 3.85 was determined for C in the general weir formula:

$$Q = CLH^{3/2}$$

where Q = total discharge

C = an empirical coefficient

L = length of weir

and H = total head (observed head plus velocity head)

Spillway Gate

The selection of type of gate for McNary spillway was difficult because it was necessary to provide protection for downstream migrating fish (primar fingerling salmon) from extreme pressure changes as they pass through the spillway. Consideration was given to both radial and vertical-lift gates divi ed into two sections so that flow could be passed between the sections durin passage of fish downstream. Serious damage of the crest and piers just dov stream from the gate slots had been experienced at Bonneville Dam with single section vertical-lift gates. Consequently, model tests were made on two-section radial gate. The tests showed that with the lower section on se and the upper section raised to pass flow between the sections, flow impinge

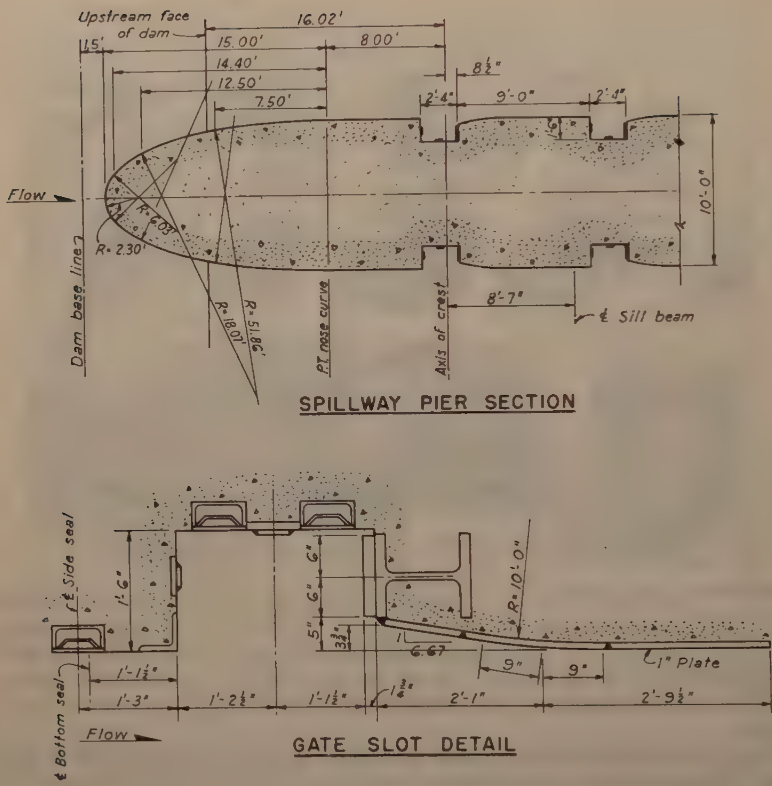
FIG. 2

on the lower gate arms and trunnion supports. As a result, the two-section vertical-lift gate shown in Fig. 2 was selected. The gate slots were designed with an offset to prevent cavitation damage to the crest and piers like that which occurred at Bonneville. Two 24-in. vents were placed in each pier to aerate the nappes when flow was passed between the upper and lower gate sections.

The shapes of the piers and gate slots are shown in Fig. 3. The 10-ft wide and airfoil-shaped pier nose cause a minimum of wave formation and vortex action. The upstream emergency gate slots and downstream operating gate slots (see Fig. 3) were offset on the downstream edge of the slots 3-3/4 in. followed by a 25-in. tapering of the side of the pier to eliminate cavitation. Characteristics for McNary spillway, in which values of H are plotted against Q, C, and K_p, are shown in Fig. 4. Model studies indicated that square-shaped slots developed negative pressures in the same areas in which damage had occurred in the Bonneville spillway. Pressure studies in the 1:36-scale model and in the prototype indicate that cavitation pressures will not occur downstream from the tapered gate slots.

Prototype Piezometers

A total of 22 piezometers were installed in bay 16 of McNary spillway; 11 of these piezometers were installed in the crest, 8 were installed in the left pier, and 3 were installed in a baffle pier. Pressures were observed with various spillway gate settings in June and July 1954 by representatives of the Walla Walla District.(3) The general pressure conditions at the spillway crest and piers, with normal pool, were satisfactory for all gate openings including free flow over the crest. Good conformity between model and prototype pressures was obtained for the crest piezometers and fair conformity for the pier piezometers. The lowest pressure observed on the crest was 7.2 ft of water approximately 30 ft downstream from the gate seal. It



McNARY SPILLWAY FIG. 3

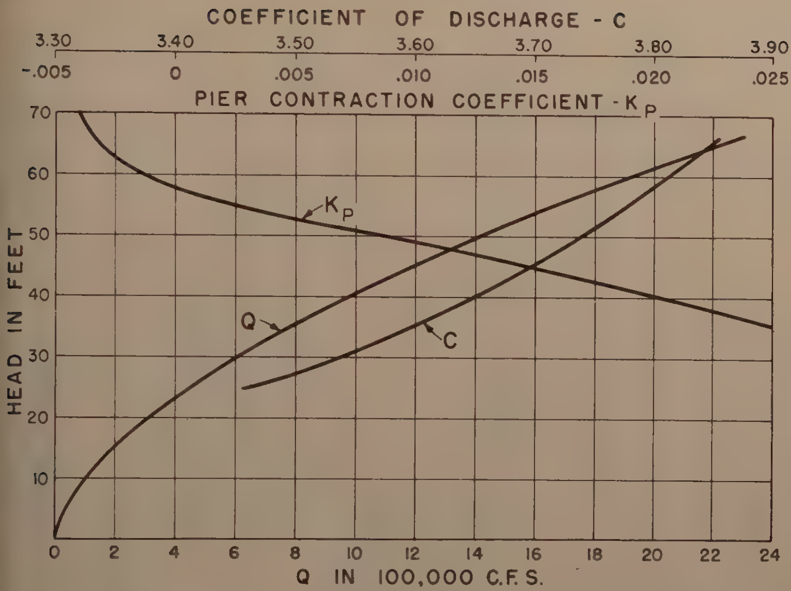
occurred with a 44.6-ft head on the crest with an opening of 10 ft under the lower section of gate. The lowest pier pressure was -1.7 ft of water with a 2.0-ft gate opening with the same head.

Chief Joseph Spillway

The spillway of Chief Joseph Dam is a concrete, gravity, ogee section wit 19 bays, 40.0 ft wide, separated by piers 9.0 ft wide. The drop from the cre at Elev. 901.5 to the stilling basin floor is 158.5 ft. The spillway is designe to pass 1,250,000 cfs with a maximum head of 55.4 ft. The flood of record a Chief Joseph is 740,000 cfs. Because fish passing facilities were not to be provided at Chief Joseph Dam, tainter gates were the only type considered i design.

Initial Design

Initial plans for the spillway provided for 21 bays, 40.0 ft wide, that woul pass the design discharge of 1,250,000 cfs under a head of 53.4 ft. Piers,



McNARY SPILLWAY CHARACTERISTICS FIG. 4

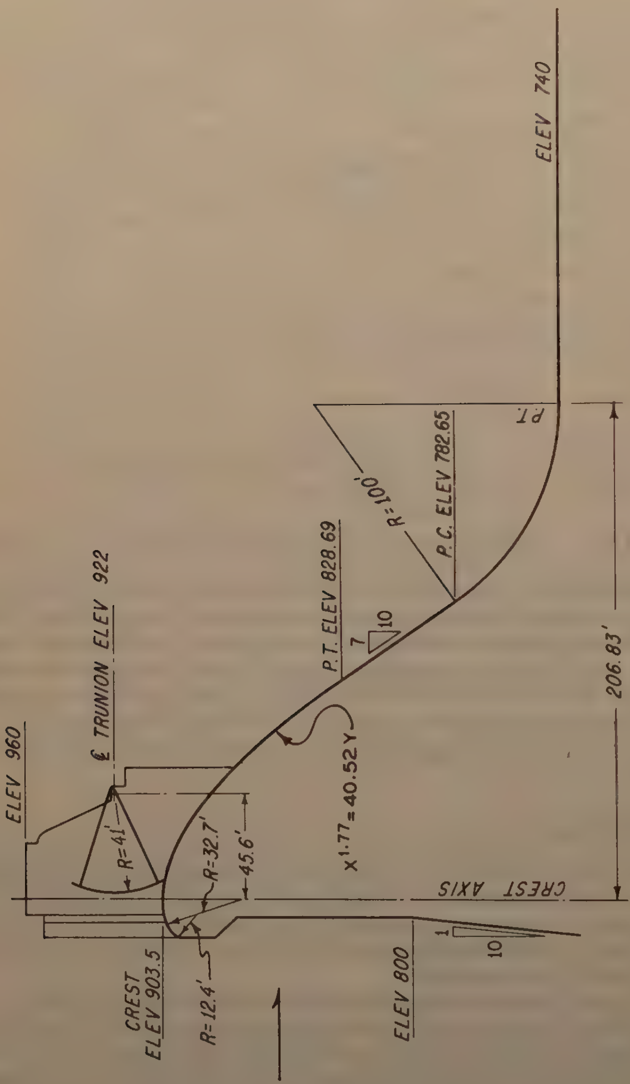
9.0 ft wide, supported 40.0-ft-wide by 37.9-ft-high tainter gates to control power pool elevations while passing discharges of 940,000 cfs and less. Thus, the gross length of the spillway in the initial design was 1020 ft and the net length was 840 ft. The spillway shape downstream from the crest was based on the equation: $x^{1.77} = 40.52y$. The initial design of the spillway is shown in Fig. 5. Attention is invited to the shape of the weir upstream from the crest axis and the location of the gate trunnions. Both of these features were subject to revision.

Hydraulic Model Studies

The initial design was studied in a 1:33-scale model of three spillway bays in a glass flume at the Bonneville Hydraulic Laboratory.[4] Outstanding design features subject to revision during the model studies of the 21-bay crest and piers (initial design) included the shape of the weir, the shape of the piers and the location of the gate trunnions.

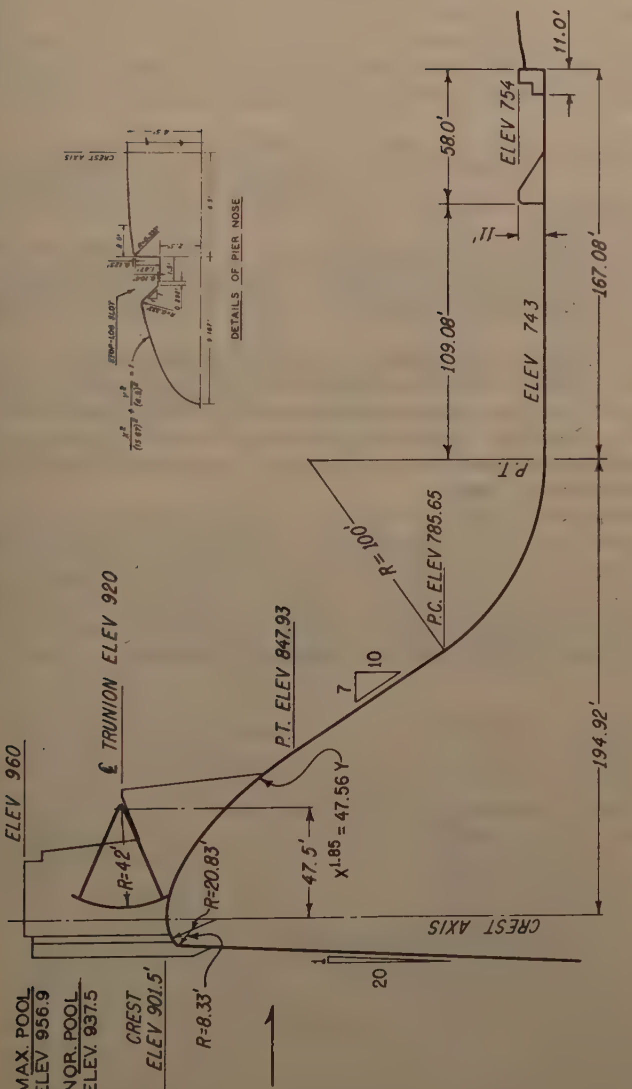
Results of the model tests revealed that the spillway design flow of 1,250,000 cfs could be passed through the 21-bay spillway under a head of 52.2 ft, whereas the computed value for the required head was 53.4 ft. Elliptically-shaped pier noses caused turbulent flow down the spillway chute during ungated discharges greater than 800,000 cfs and standing waves that overtopped steps on the piers during the spillway design flow.

Design studies by the Seattle District based on the results of the model studies indicated that a net saving of approximately $1,300,000 could be realized by a reduction of the spillway length. To accomplish this, one gate was eliminated at each end of the spillway, the crest elevation was lowered 2.0 ft to Elev. 901.5, and the crest was under-designed for 75 per cent of the maximum head of 55.4 ft resulting in a design head of 41.6 ft.



INITIAL DESIGN

CHIEF JOSEPH SPILLWAY FIG. 5



ADOPTED DESIGN

CHIEF JOSEPH SPILLWAY

FIG. 6

19-Bay Spillway

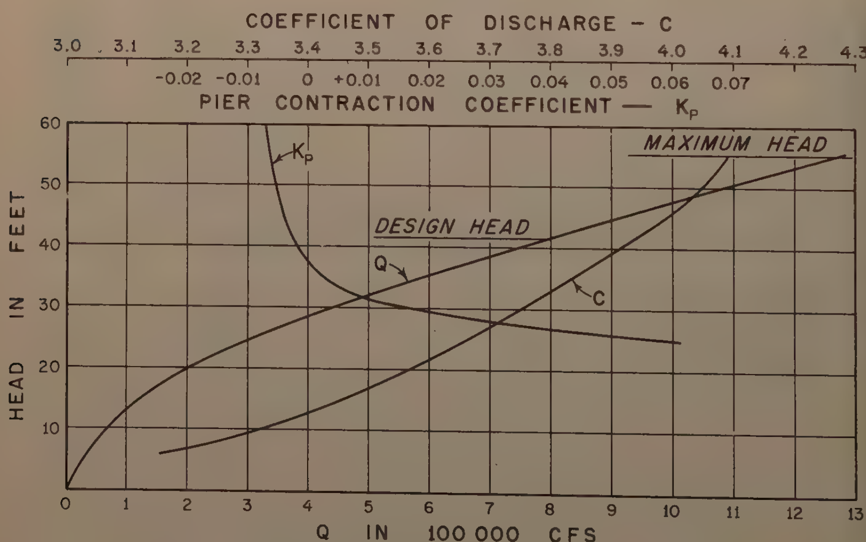
Design features of the 19-bay spillway that were still subject to revision depending upon results of the model studies included the shape of the weir, stop-log slots, and downstream edges of the piers and the location of the gate trunnions. The spillway weir adopted for design, shown in Fig. 6, was tested in the model first without piers and later with the piers installed so that the values of discharge coefficients and pier contraction coefficients could be determined.

Tests were made with several heads on the weir at approximately 5-ft intervals in the lower range and at 1-ft intervals as the maximum head of 55.4 ft was approached. Coefficients of discharge were then computed from the equation, $Q = CLH^{3/2}$.

Average values of H and C are plotted in Fig. 7. It will be noted that average values of C varied from about 3.15 with a 5.5-ft head on the crest to 3.94 at the design head of 41.6 ft and increased to approximately 4.09 at the maximum head of 55.4 ft.

Pressures on the spillway crest were nearly atmospheric during the test that reproduced a head of 42.1 ft. At a head of 55.0 ft, a pressure of -23 ft of water occurred at the piezometer just downstream from the spring point on upstream face of the spillway. Pressures were negative for a distance of 50 ft downstream from the crest axis as a result of designing the spillway for 75 per cent of the maximum head.

Tests in the model with piers were made with several variations of pier noses and gate slots in an effort to eliminate the need of armor plating downstream from the gate slots to protect the piers from cavitational damage. However, efforts to eliminate cavitational pressures with the maximum design flow of 1,250,000 cfs were unsuccessful and the pier noses and gate slot shown in Fig. 6 were adopted. Armor plate was not used on the piers. It was



CHIEF JOSEPH SPILLWAY CHARACTERISTICS
FIG.7

considered that it would be more economical to repair the piers in case of any damage from high flows of infrequent occurrence.

Determination of the head-discharge relationships (Fig. 7) that resulted during free flow over the adopted design crest with piers permitted computation of pier contraction coefficients from the weir equation: $Q = C\,(L - K_p nH)\,H^{3/2}$. The curve of K_p values plotted against head in Fig. 7 indicates that pier contraction coefficients were negative at heads in excess of approximately 38 ft and decreased to a value of -0.008 at a head of 56 ft. This indicated that the discharge per foot of crest was greater with the piers installed than without piers at heads in excess of approximately 38 ft because of the negative values of the pier coefficient.

Pressures on the ogee were higher with the piers installed than without piers as shown in Fig. 8. Nevertheless, negative pressures existed with piers near the pier line with a discharge of 940,000 cfs, which is the minimum ungated flow, and with larger flows. The lowest observed pressure on the ogee, which occurred during the design discharge, was -9 ft of water at the piezometer just downstream from the crest and 1 ft from the edge of the pier. The pressure was -5 ft at the corresponding piezometer on the center line of the bay for the same discharge.

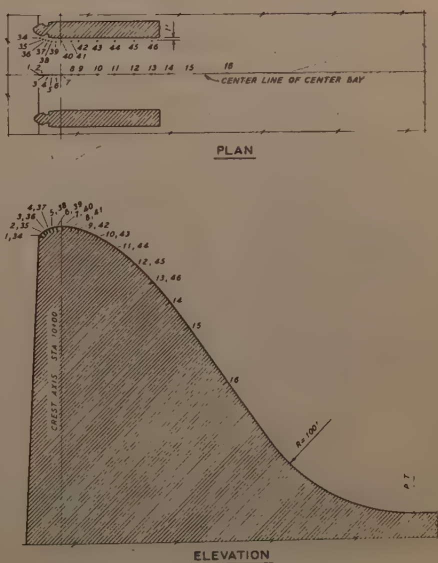
PRESSURES ON CREST
19-BAY SPILLWAY - FINAL DESIGN
FREE FLOW

Piez. No.	Elev	Without Piers			With Type 2 Piers	
		Total Head on Crest				
		54.3	49.1	43.1	54.0*	46.0**
		Head on Piezometer in Feet of Water				
1	896.5	-23	-9	1	-1	10
2	898.1	-10	-1	5	-4	13
3	899.4	-19	-8	-1	-3	7
4	900.2	-17	-7	0	-3	5
5	900.9	-13	-5	1	-2	7
6	901.4	-13	-6	0	-4	4
7	901.5	-13	-7	-1	-5	3
8	900.9	-8	-3	1	-3	3
9	900.0	-9	-4	0	-5	2
10	896.1	-7	-3	0	-4	1
11	890.1	-5	-2	0	-3	0
12	882.2	-3	-1	0	-2	-1
13	872.3	-2	-1	0	-1	0
14	860.5	1	-1	2	1	2
15	847.0	—	—	—	—	—
16	816.5	11	9	8	7	9
34	896.5	-22	-9	1	17	22
35	898.1	-8	0	7	18	22
36	899.4	-14	-6	2	13	18
37	900.2	-15	-6	1	2	10
38	900.9	-12	-5	2	-2	-1
39	901.4	-13	-6	0	-7	-5
40	901.5	-12	-6	0	-9	1
41	900.9	-8	-4	1	-2	4
42	900.0	-8	-4	1	-3	3
43	896.1	-7	-3	0	-3	2
44	890.1	-4	-2	0	-1	2
45	882.2	-3	-1	0	-1	1
46	872.3	-2	-1	0	1	2

* Discharge 1 250 000 CFS

** Discharge 940 000 CFS

— Piezometer out of order



PIEZOMETER LOCATIONS

CHIEF JOSEPH SPILLWAY FIG. 8

Pressures that existed on the piers during spillway discharges of 1,250,000 cfs and 940,000 cfs are shown in Fig. 9. Pressures that were sufficiently negative to produce possible cavitation were observed adjacent to the stop-log slots during design discharge. At the remaining piezometers, pressures generally were atmospheric or greater.

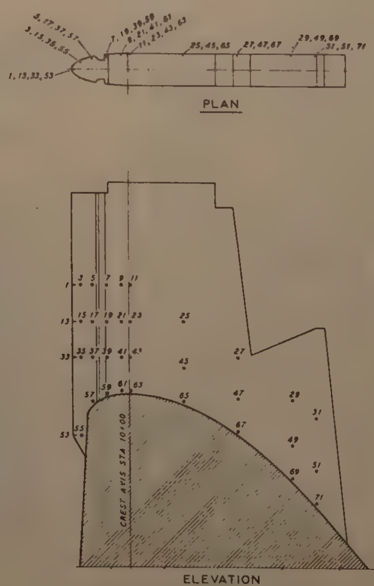
Prototype Tests

Eighteen piezometers were installed in spillway bay 9 and 5 were installed in the left adjacent pier at Chief Joseph Dam. Four pressure cell recesses were provided adjacent to four of the piezometers located in the ogee along the center line of the bay just downstream from the crest axis. These cells will be used to measure instantaneous pressures on the crest and to check pressure data obtained from the piezometers.

Prototype tests to obtain pressure data were conducted by the Waterways Experiment Station for the U. S. Army Engineer District, Seattle. Forty-four tests were made at Chief Joseph Dam in June and July 1956. Data were obtained at heads ranging from 0.8 to 1.1 times the design head (41.6 ft) in bay 9 at full and partial gate openings. Results(5) showed that good correlation existed between the model and prototype data and that negative pressures,

PRESSURES ON SPILLWAY PIERS
19-BAY SPILLWAY WITH TYPE 2 PIERS
FREE FLOW

Piezometer		Discharge in CFS	
		1 250 000	940 000
Number	Elev	Head on Piezometer in Feet of Water	
1	931.5	26	17
3	931.5	13	10
5	931.5	12	8
7	931.5	13	9
9	931.5	7	5
11	931.5	9	5
13	921.5	35	27
15	921.5	19	16
17	921.5	13	12
19	921.5	18	15
21	921.5	10	9
23	921.5	12	10
25	921.5	7	5
27	911.5	4	3
29	900.0	3	2
31	895.0	2	2
33	911.5	44	36
35	911.5	25	23
37	911.5	19	17
39	911.5	21	20
41	911.5	8	10
43	911.5	10	11
45	907.4	7	8
47	900.1	1	3
49	887.4	1	2
51	880.5	1	2
53	890.1	57	51
55	890.1	59	55
57	899.7	18	20
59	901.5	-28	-15
61	901.9	-21	-9
63	902.5	-6	2
65	898.4	0	4
67	891.1	-1	4
69	878.4	1	2
71	871.5	1	1



PIEZOMETER LOCATIONS

CHIEF JOSEPH SPILLWAY FIG. 9

produced when the design head was exceeded, were as indicated by the model. The results also provide further justification for the use of model studies in the design of underdesigned spillways.

The stability of the spillway must be considered when the crest is designed for 75 per cent of the maximum head. However, costs saved in initial construction more than offset the cost of infrequent repairs resulting from damage due to negative pressures.

The Dalles Spillway

The design discharge for The Dalles spillway is 2,290,000 cfs. The size of the spillway was determined by the criteria that 1,050,000 cfs, the regulated 1894 flood of 1,240,000 cfs (flood of record), be passed at a normal pool elevation of 160 ft, mean sea level. Also that sufficient capacity be provided to prevent overtopping of the structures, with a reasonable freeboard remaining, on occurrence of the spillway design flood. In order that the height of the spillway gates should not exceed approximately 40 ft, which was considered the maximum height for a single gate by representatives of the various fish agencies (for safe passage of fingerling salmon under the gate), the spillway crest was set at Elev. 121.0. The width of the spillway bays and piers was established as 50 and 10 ft, respectively.

Spillway Design

The spillway initially was designed with 27 bays. The model studies indicated that the spillway capacity was about 6 per cent greater than assumed in the design computations. This, plus subsequent revisions in design criteria, which included redesigning the crest for 75 per cent of the maximum head and increasing maximum pool to Elev. 184, resulted in the adoption of a 23-bay spillway. The 23-bay spillway will pass the regulated flood of record (1,050,000 cfs) with normal pool at Elev. 160.

The spillway crest was designed for a head of 46.0 ft or 75 per cent of the maximum computed head of 61.3 ft using a standard crest shape. Details of the crest shape are shown in Fig. 10. The maximum head of 61.3 ft includes velocity head of approach and results in a required computed energy grade line at Elev. 182.3 just upstream from the spillway for the design discharge of 2,290,000 cfs. The upstream face of the spillway is vertical above Elev. 100 and on a slope of 2 vertical on 1 horizontal below Elev. 100, as required for structural stability. Upstream from the spillway axis, the crest is formed by arcs of two circular curves having radii of 23.0 and 9.2 ft; downstream from the axis it conforms to the equation, $x^{1.85} = 51.805y$.

Spillway Gates

Control of the discharge over the spillway is by tainter gates 50 ft wide by 40 ft high, measured from crest to top of gates. This provides 2 ft of freeboard from the top of a closed gate to normal pool elevation. Although split gates were required for fish passage at McNary with a normal head of 49 ft on the spillway crest, a single section radial-type gate was approved for The Dalles with a normal head of 39 ft. During the design of The Dalles spillway, pressure studies were made on fingerling salmon at the Bonneville Hydraulic Laboratory. The studies showed that fingerling salmon could be passed

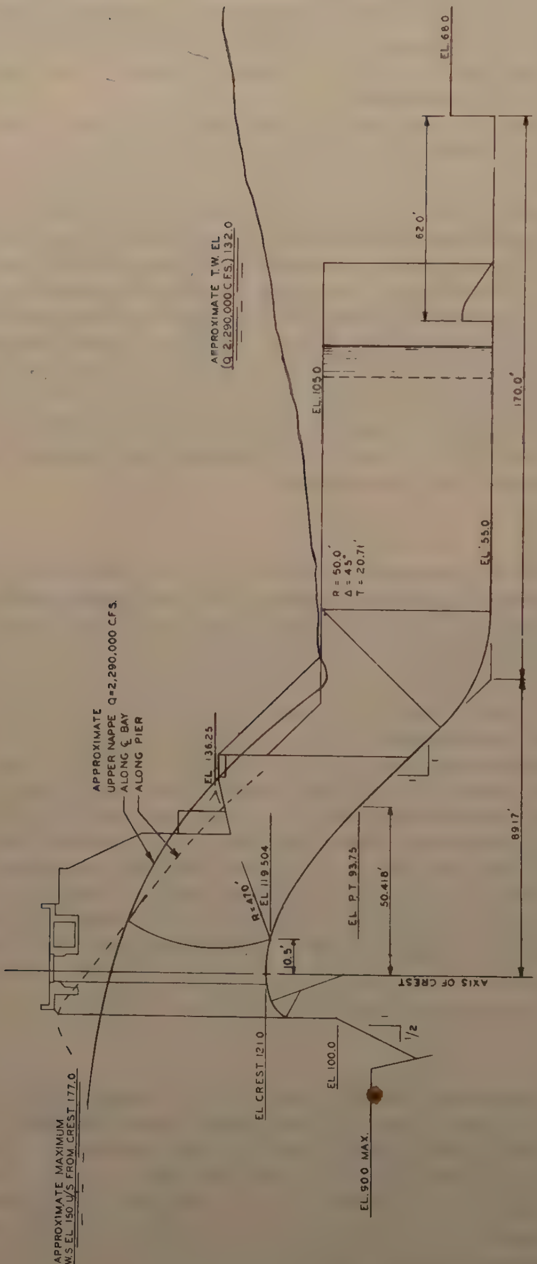
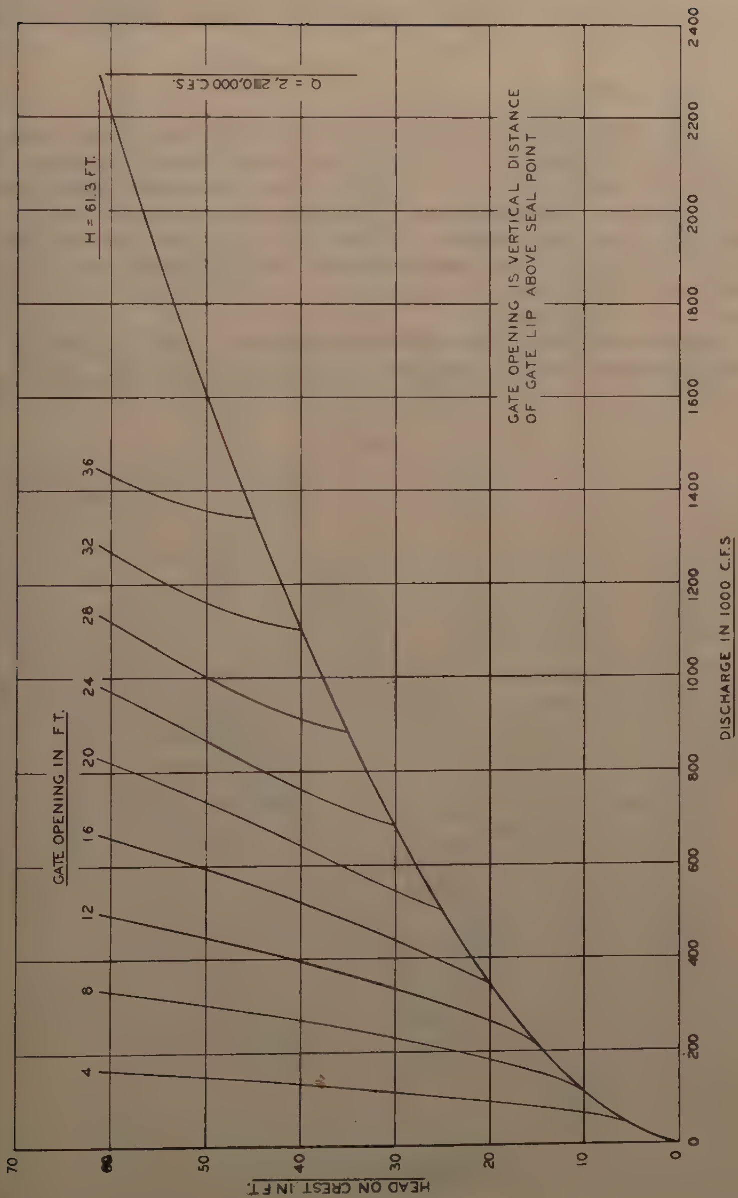


FIG. 10

THE DALLES SPILLWAY SECTION



SPILLWAY RATING CURVES

THE DALLES SPILLWAY

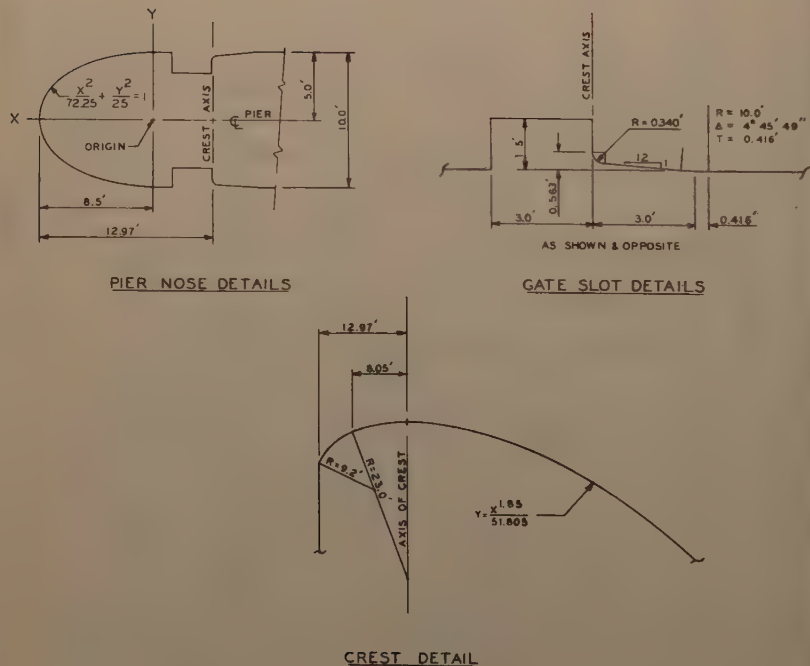
FIG. 11

downstream under spillway gates, without injury to the fish, at much higher heads than had been considered safe for fish passage prior to the tests. Because of economy in original cost and operation, tainter gates are now being used in the design of spillways on all new Corps of Engineers projects on the Columbia and Snake Rivers. Rating curves for the spillway gates are shown in Fig. 11.

Hydraulic Model Studies

Details of the upstream nose of the piers and the emergency gate slots are shown in Fig. 12. The nose of the piers is in the shape of a 1.7:1 ellipse. The gate slots were designed with a 1 to 12 offset on the downstream side to prevent cavitation damage. Studies in the 1:36-scale model indicated that all pressures on the piers were positive for flows up to 1,050,000 cfs with both gated and free flow. A low negative pressure of -17 ft of water was observed in the offset downstream from the gate slot and 1.0 ft above the crest with the design discharge of 2,290,000 cfs.

Water-surface and pressure profiles on the spillway crest observed in the model with a simulated flow of 2,290,000 cfs are shown in Fig. 13. The lowest



THE DALLES SPILLWAY FIG. 12

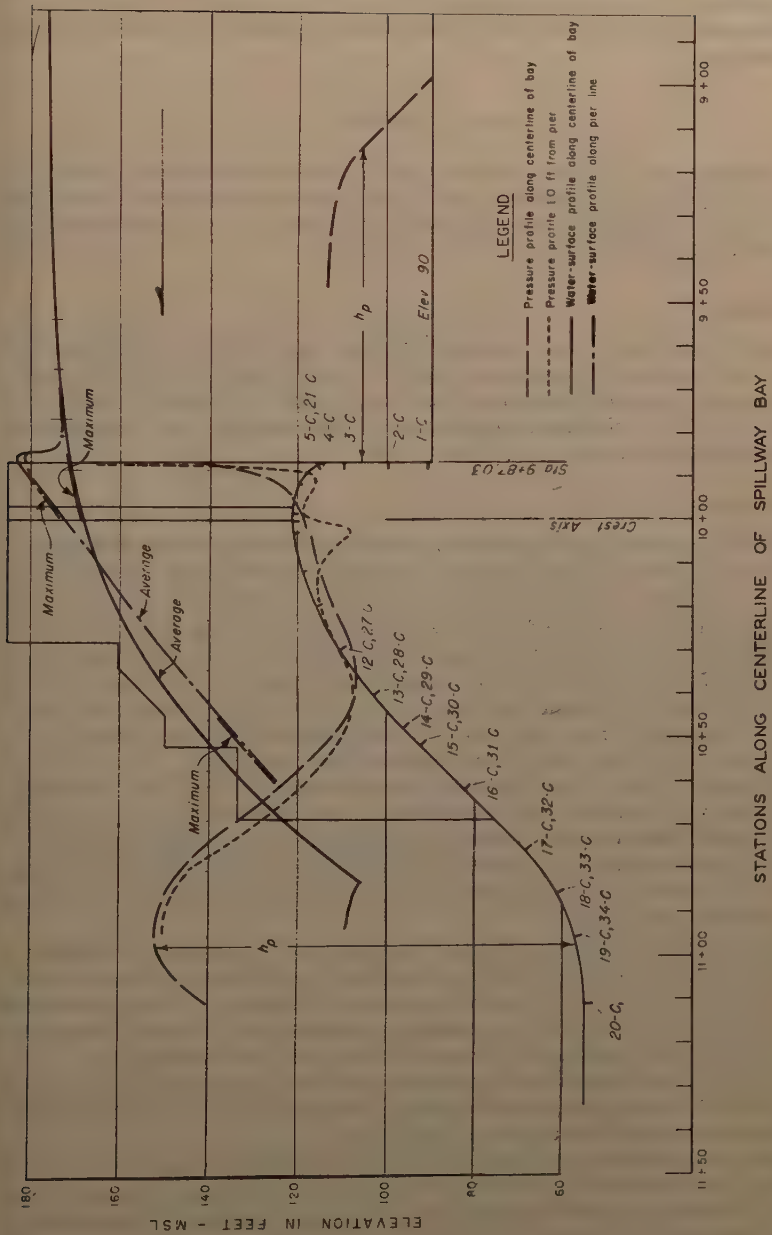


FIG. 13

THE DALLES SPILLWAY

pressure observed on the crest, -13 ft of water, was on a piezometer located just downstream from the crest center line and 1.0 ft from the pier. This conforms closely to the performance of Chief Joseph Spillway, also designed for 75 per cent of the maximum head.

No piezometers were installed in The Dalles spillway for observing prototype pressures.

Detroit Spillway

The spillway portion of Detroit Dam is a concrete ogee section with a maximum drop of 371 ft from the crest to the stilling basin floor. The spillway is designed to pass a maximum flow of 176,000 cfs. The maximum flood of record is about 61,300 cfs.

Initial Design

Initially the spillway was designed with four bays 64 ft wide with piers 13 ft thick. Flow was to be controlled with 28-ft-high tainter gates. The face of the spillway downstream from the crest was designed to conform to the equation, $x^2 = 78y$. Upstream from the crest, the spillway was shaped to two curves with radii of 13.68 and 7.17 ft.

Adopted Design

During the course of model studies performed at the Waterways Experiment Station,(6) revisions were made in design and the number and size of the spillway crest gates and the alignment of the ogee section were changed. The adopted design has six tainter gates, 42 ft wide by 28 ft high separated by 8.5-ft-wide piers. Gross length of the spillway is 294.5 ft. The adopted spillway shape, designed for 100 per cent of the maximum head on the crest of 33.0 ft and shown in Fig. 14, is based on the equation $x^{1.85} = 36y$ downstream from the crest. Upstream from the crest, the compound curve with radii of 15.0 and 6.0 ft forms an overhang above the face of the spillway.

Studies were made in the model to investigate the effect of the overhang. Measurements of head-discharge relations, and water-surface and pressure profiles showed identical results with and without the area filled below the overhang. The similarity of results is attributed to the fact that the overhang extends 15 ft below crest level which apparently is far enough to prevent any change in flow conditions.

The semicircular shaped pier nose was selected for the adopted design. Since repairs to the spillway gates can be made when the pool is below the elevation of the crest, no provision was made for stop logs. Without stop-log slots, the semicircular pier nose operates satisfactorily. Model studies with the initial design showed slightly higher values for C in the equation $Q = CLH^{3/2}$ for the semicircular pier nose as compared with a streamlined pier nose in the lower range of discharges. However, the value of C was 3.70 for both shapes with a discharge of 157,600 cfs.

Pressure observations in the model with the adopted design and with maximum pool for conditions of free and controlled flow indicated that only slight negative pressures existed with the gates at partial opening. The maximum negative pressure observed on the crest was -1.0 ft of water and occurred for a gate opening of 12 ft. With free overflow, pressures in the same area were all positive at the maximum discharge.

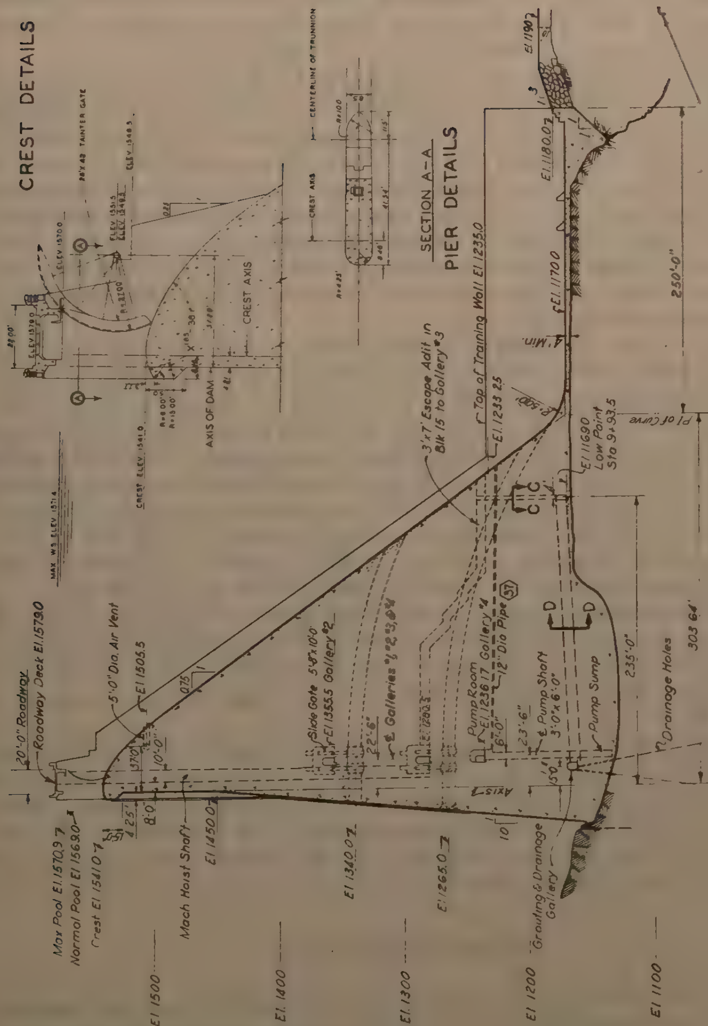


FIG. 14

DETROIT SPILLWAY SECTION

Cougar Spillway

The general plan of the Cougar Dam Project is shown in Fig. 15. The spillway is located in a deep, narrow cut around the right abutment of the rock-fill dam. Flow from the spillway will be discharged into the quarry area on the right bank downstream from the rock-fill dam.

The discharge capacity of the spillway is computed to be 74,800 cfs with maximum pool at Elev. 1699. Maximum design flood is 86,000 cfs. Flow over the spillway will occur only during extremely high floods.

Two 40-ft-wide spillway bays, separated by a 9-ft-wide pier, were selected as the minimum number of bays consistent with economical tainter gate size and approach channel excavation. The approach channel at the spillway structure is at Elev. 1635 (see Fig. 15) to provide a spillway weir height equal to approximately one-half the maximum head. The channel continues upstream on a gentle curve and with a widening cross section until it intersects the natural topography. The floor of the channel is on a one per cent slope for drainage. Head losses in this channel have been computed to be 0.6 ft, maximum velocity 11 fps, and super-elevation of water surface approximately 1 ft for the maximum discharge.

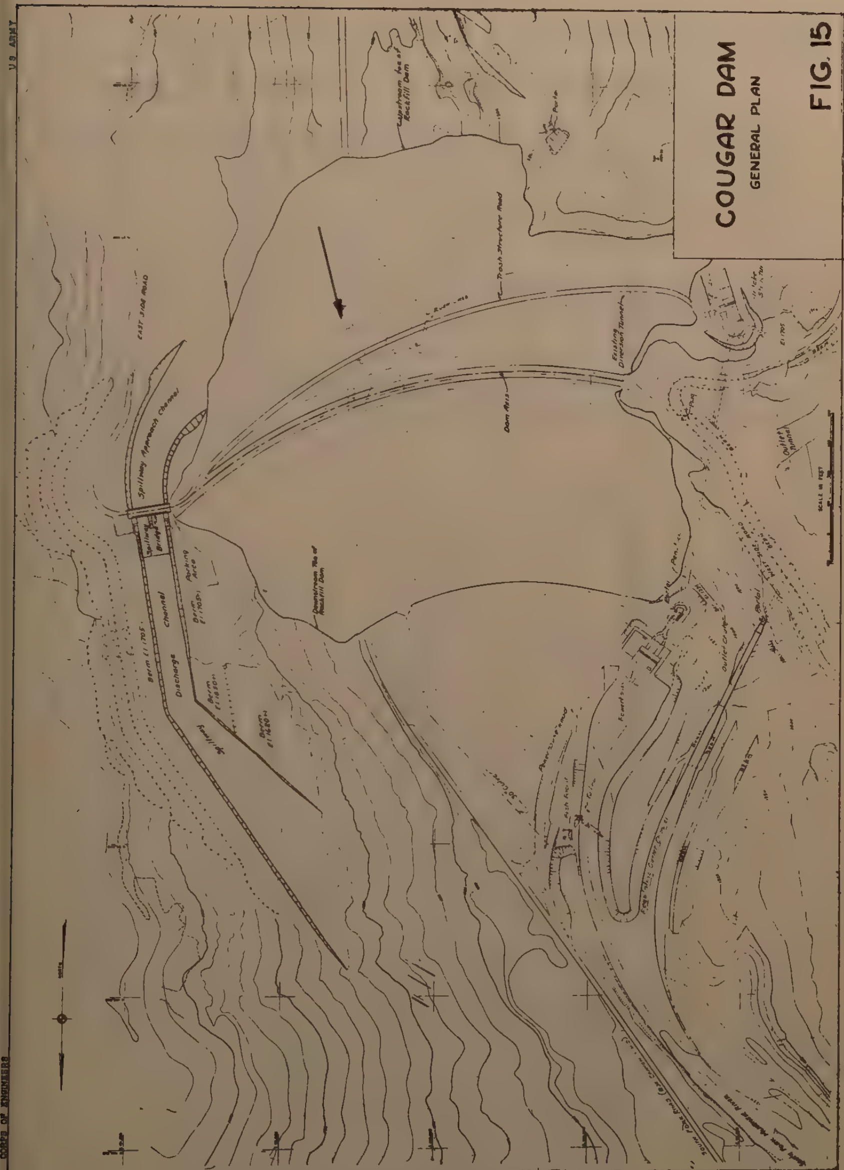
A section through the spillway crest is shown in Fig. 16. The crest was designed for 100 per cent of the maximum head of 41.65 ft. Consideration was given to an underdesigned crest of 75 per cent of the maximum head. However, it was not economical to lower the approach channel floor to provide a weir height greater than the maximum head on the crest. It was believed that unless the height of the weir exceeded the maximum head, an underdesigned crest might be subject to damaging vibrations from the overfalling nappe. The maximum head of 41.65 ft includes velocity of approach, 1.9 ft, and is the difference between an energy grade line at Elev. 1698.4, just upstream from the spillway for a discharge of 74,800 cfs and crest at Elev. 1656.75.

The pier nose separating the two spillways is semicircular in shape. Spillway discharge coefficients and pier and abutment coefficients were taken from the Engineering Manual.[1] The two tainter gates that will control the flow through the spillway are each 40 ft wide by 43.5 ft high with the top of the gates at Elev. 1699.

Spillway rating curves were computed for free overflow and for partial gate openings. The rating curve for free overflow was computed from the equation, $Q = C\,[L - 2\,(nK_p + K_a)\,H]\,H^{3/2}$, using $C = 3.93$ at maximum head. Values of the pier and abutment contractions used were $K_p = -0.01$ and $K_a = 0.12$, respectively, at maximum head. Rating curves for partial-gate openings were computed from the equation, $Q = CA\sqrt{2gh}$.

ACKNOWLEDGMENTS

The author wishes to acknowledge the use of material from the Portland District, Corps of Engineers, U. S. Army, in the preparation of this paper. Particular acknowledgment is given to staff members of the Hydraulic Design and Laboratory Section for assistance in preparation and review of the paper.



COUGAR DAM
GENERAL PLAN
FIG. 15

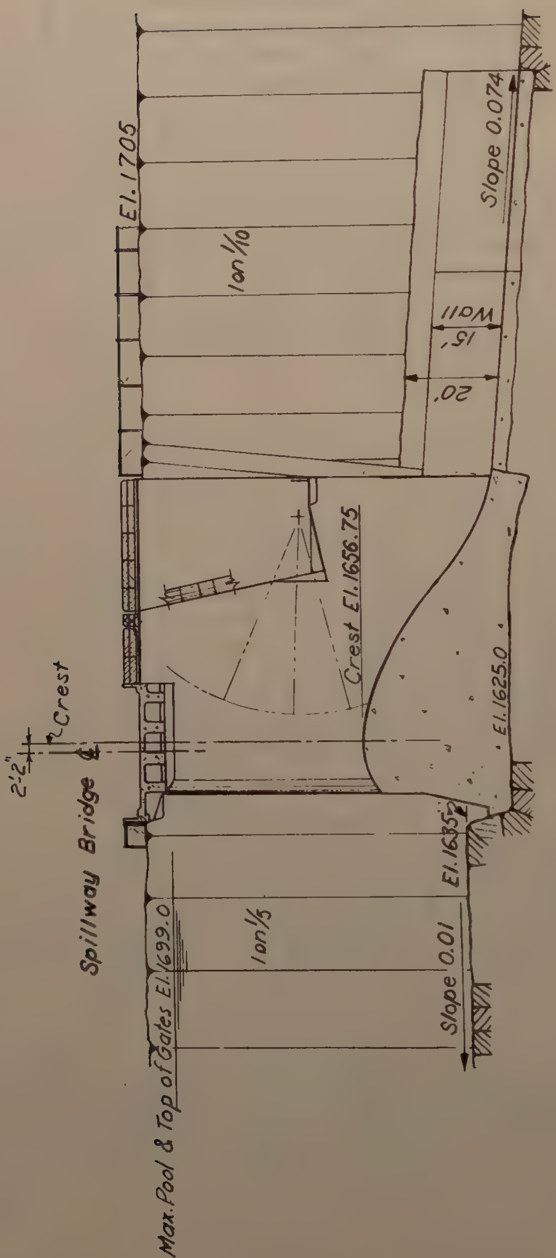


FIG. 16

COUGAR SPILLWAY SECTION

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DIVINING RODS VS. HYDROLOGIC DATA AND RESEARCH[a]

Closure by W. B. Langbein

W. B. LANGBEIN,[1] F. ASCE.—The comments by Drs. Chow and Yevdjevich are greatly valued; they provide very pertinent amplification of the objectives of the paper—a plea for more research and a wider public use of hydrologic data.

As Dr. Chow states, the divining rod like all mystical decision-makers "constitutes indeed a stumbling block to the progress of technology and to the development of any scientific program."

The writer heartily endorses Dr. Yevdjevich's appeal for greater efficiency in hydrologic research and his observation that the collection of hydrologic data today should anticipate water-resources development two or three decades hence.

Growing awareness of these principles is leading toward good progress in improving the basic-data programs and in stimulating research. The outlook appears bright.

I think Mr. Dittbrenner, a professed hydraulic engineer-dowser, missed the significance of my query "can we be scientific about witchery?" He wrongly imputes to me a negative answer to this question. On the contrary, I made clear that a scientific approach to witching is possible even though perhaps leads along a different course from the kinds of explanations Mr. Dittbrenner has in mind. A book published since the writing of my article ("Water-witching, U.S.A.", by E. Z. Vogt and Ray Hyman, Univ. of Chic. Press, 1959) provides excellent evidence of the conditions motivating the use of the divining rod. The evidence presented shows that water witching prevails as a ritual practiced in the face of anxiety, and uncertainty.

a. Proc. Paper 1809, October, 1958, by W. A. Langbein.
1. Hydr. Engr., U. S. Geological Survey, Washington 25, D. C.

QUEUING THEORY AND WATER STORAGE[a]

Closure by W. B. Langbein

W. B. LANGBEIN,[1] F. ASCE.—Mr. B. W. Gould's analysis of the reservoir problem where discharge is a linear function of the storage on hand shows different results depending on what boundary conditions are taken for the routing equation. However, only one case meets the conditions in usual practice and that is the one given in the paper.

In brief, Mr. Gould presents three cases for which the routing equations used to develop the equation of storage are as follows:

Case I (as given in original paper)

$$D_x = \frac{k}{1+k} I_x + \frac{1}{1+k} D_{x-1}$$

In this case, the discharge in time interval x is a weighted average of the contemporaneous inflow and the discharge in the preceding interval. This is a feasible form of regulation. No forecasts are involved. Mr. Gould's remark "This type of control would be difficult to achieve in practice" seems rather strained in view of the simplicity of this equation. One may assert the contrary.

Case II

$$D_x = \frac{2k}{2+k} I_{x-\frac{1}{2}} + \frac{2-k}{2+k} D_{x-1}$$

In case II the discharge at time x is a weighted average of the inflow one-half unit of time before x, and the discharge one unit of time before x.

Case III

$$D_x = kI_{x-1} + (1-k) D_{x-1}$$

In case III the discharge from the reservoir at time x is a weighted average of the inflow to and the discharge from the reservoir in the time interval preceding period x.

Case I is expressed in terms of the concurrent inflow, case II in terms of inflow one-half interval previous, and case III, one full interval previous.

It should be noted that the values of k for these three conditions also differ. For case I, k is any value greater than 0; for case II, k cannot exceed 2; for

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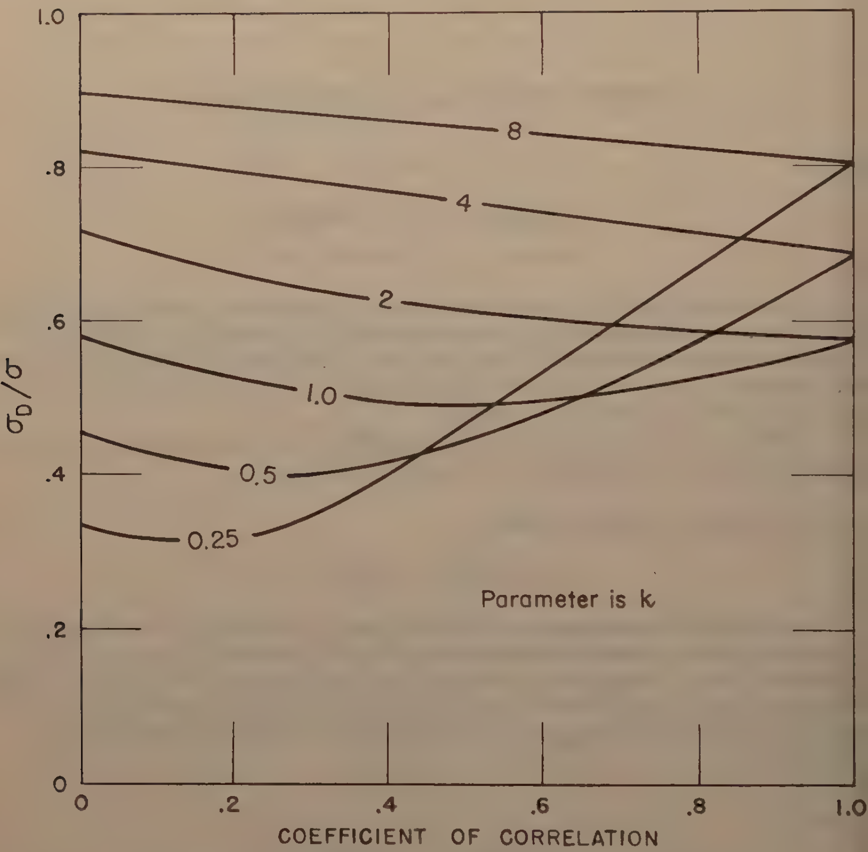
a. Proc. Paper 1811, October, 1958, by W. B. Langbein.

1. Hydr. Engr., U. S. Geological Survey, Washington, D. C.



Effect of forecast of inflow one time unit ahead, in reducing variability of discharge.

case III, k cannot exceed 1. There appear to be no physical reason why values of k are so limited. Moreover, for these indicated limiting values of k, the variability of the discharges equals that of the inflow, a condition for which no storage should be needed. Yet the equations for cases II and III show that storage is needed when k equals 2 and 1, respectively. No storage is indicated for case I when k is infinity. The reason that storage is needed for cases II and III even though the discharges equal the outflows is that the discharges are delayed one-half time interval for case II and a full time interval for case III.

One might picture a storage reservoir in which this year's discharge is made successively equal to the preceding year's inflow; the time advantage might afford opportunity for planning, but, in general, such a plan must be considered trivial in practice. In this mathematical model there is no increase in the minimum flow, nor decapitation of floods.

The case offered in the original article appears to be the only practical one and indeed those mentioned by Gould were originally considered and rejected.

However, these original exercises did produce one result not presented but which might be mentioned here. It may be noted that for a given rate of regulated flow, b, storage requirement is least for case I, and greatest for case III. Cases II and III are related to inflows going backward in time. But suppose there are forecasts providing information about inflows in prospect. Would this information increase the value of the minimum regulated flow or alternately decrease the amount of storage?

To answer this question, a plan of operation was assumed such that the discharge at any time is a linear function of the inflow in prospect during the next interval ahead, as well as the current inflow and the storage on hand. It was further assumed that the inflow in prospect can be weighted in accord with the coefficient of correlation.

The attached illustration shows the results for increasing values of the correlation coefficient of the forecast.

The abscissa is the coefficient of correlation of the forecast; the ordinate is the standard deviation of the discharges in ratio to the standard deviation of the inflows. It is desired that the variability of the discharges be a minimum. Several curves are shown each for different values of k. The lower the value of k, the greater the "carry-over" period of the reservoir. The graphs show clearly that the smaller the reservoir, the greater is the significance of forecasts of the inflow for one time unit ahead. For larger reservoirs the utility of such short-term forecasts becomes less, and indeed there is the possibility that using such information in the scheme of regulation as described might actually vitiate the uniformity of discharges from a long-period reservoir. One might add that to be useful, the longer the detention period, the farther ahead the forecasts must be.

INTERIM CONSIDERATION OF THE COLUMBIA RIVER ENTRANCE[a]

Discussion by R. E. Hickson

R. E. HICKSON,[1] F. ASCE.—This paper presents much data as to conditions at the mouth of the Columbia, past and present, as obtained from hydrographic surveys extending over a long period of years, the records of dredging, jetty construction, current surveys, etc. Much can be gained by a study of these data. There are, however, differences of opinion as to the influences and forces which have caused the changes that have taken place. Some of the forces and combinations of forces which are continually at work are not readily susceptible of measurement.

The author expresses his views as to causes and effect of some of the changes and adverse conditions presently obtaining, and also presents a plan of action to secure additional data as to currents, salinity, etc. This additional information will serve as a check on and add to the material presently available, much of which was presented to the Chief of Engineers in Current Survey report of 1932-33.

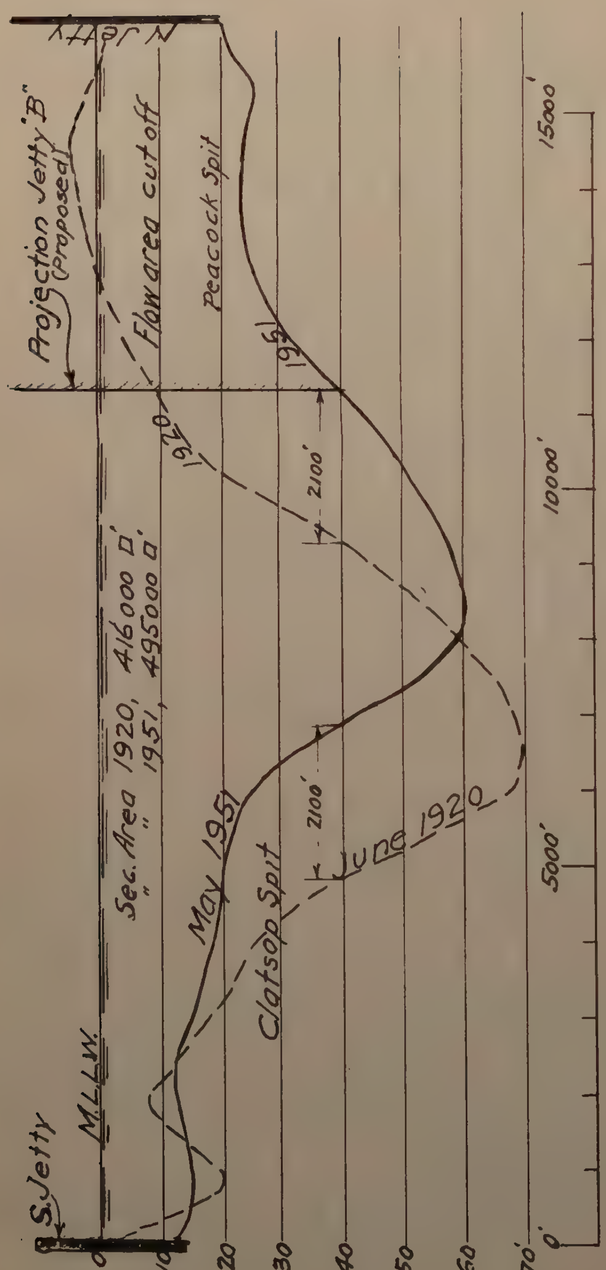
In this paper there will be presented a discussion of several of the statements of the author with which the writer does not agree. It is hoped this will tend to clarify our thinking in regard to channel improvements and maintenance, and result in a fuller understanding of the situation at mouth of the Columbia.

Near the middle of page 33 (Paper 1902) the author states: "Examination of these (condition) surveys reveals that greater than project depths prevailed since the beginning of improvement through a natural channel with length ranging from 4 to 7 miles, lying just inside the bar (and) between Clatsop Spit and Sand Island." This is entirely correct, and this exceptionally good channel prevailed for about 16 years (1916 to 1932) including the year 1927 which the author shows on his Fig. 3. It should be stated, however, that during this period, and before, the position of the channel was gradually shifting to the north due to erosion of the north bank in the bend. This erosion cut away a large sand bar, Peacock Spit, extending about 5000' to the south of Cape Disappointment, part of which bar was dry in 1920. Erosion there increased depths by 20 to 30' by 1951. Reference to sections I-I and II-II, and charts A and B herewith, shows shift of the channel to the north. From a longer range standpoint, say over a period of 60 years (1879 to 1939), the south shore of Sand Island moved north about 5300 feet.

The author apparently thinks the shifting of the channel was due entirely to building of Clatsop Spit on the south side of the channel, from littoral drift and

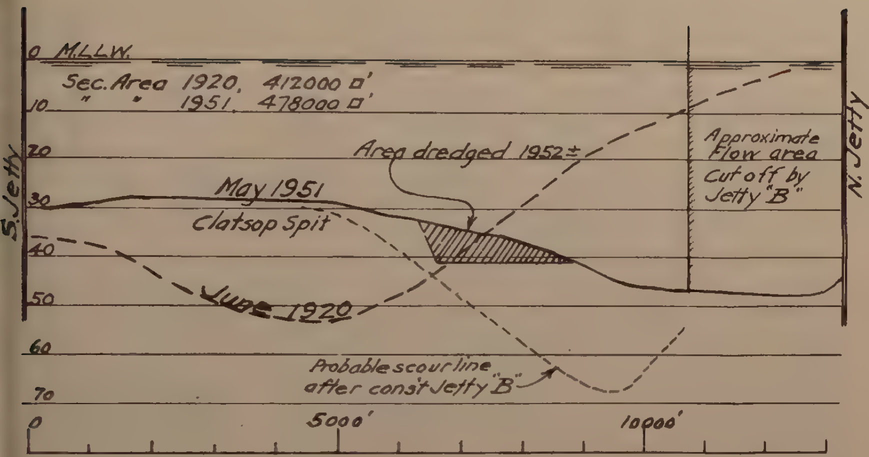
a. Proc. Paper 1902, January, 1959, by John B. Lockett.

1. Head Engr., Corps of Engrs., Portland Ore. Dist., Retired; formerly member of Committee on Tidal Hydraulics.



MOUTH COLUMBIA RIVER 1920-1951
Sections, "Knuckle" to McKenzie Head -
(Chart 'A')

Sections I-I



MOUTH COLUMBIA RIVER 1920-1951
Sections, N-S 3000' W. McKenzie Head
(Chart "B")

Sections II-II

density current deposits, but says near the bottom of page 20: "It is interesting to note that for the first time, depths in excess of 40 feet prevailed along the channel side of the North Jetty." This increase in depth was, of course, due to scour in the bend on the north side of the channel referred to above, which is the characteristic history of all streams with curved alignment. (For movement here 1903 to 1921 see Military Engineer July-Aug. 1922 p. 211 et seq.) If the north bank (in the bend) as located in, say 1920, had been composed of rock or was protected from erosion, the channel could not have shifted, velocities would have been maintained at the high figure necessary for maintenance of the good channel section obtaining at that time, and the shoal on the south side, now giving trouble, could not have been formed.

The present channel at this point is badly out of alignment and the shoal will probably persist on the south side until the alignment of 1916-32 is approached, and shifting into the bend is prevented by control works. Such control has been effected along Sand Island and in the vicinity of Jetty A, with complete success. (Chart A and B.)

The author lays considerable stress on littoral and density currents as probable causes of the shoal at Clatsop Spit. It may properly be pointed out, however, that the same littoral and density currents, whatever their effects may be, prevailed in general in the years 1916 to 1932 as exist today. Still the channel was in excellent condition through all that period, although gradually shifting to the north.

The existence of density currents in this vicinity has been known since time of the early lead line surveys, and were definitely found during the current survey of 1932. At the Columbia entrance, however, density currents as such are evident for only about 1-1/2 hours at first of the flood tide, after which the currents are all upstream (flood) from surface to bottom and maximum velocities are at about 1/3 depth at strength of the flood, rather than on the bottom.

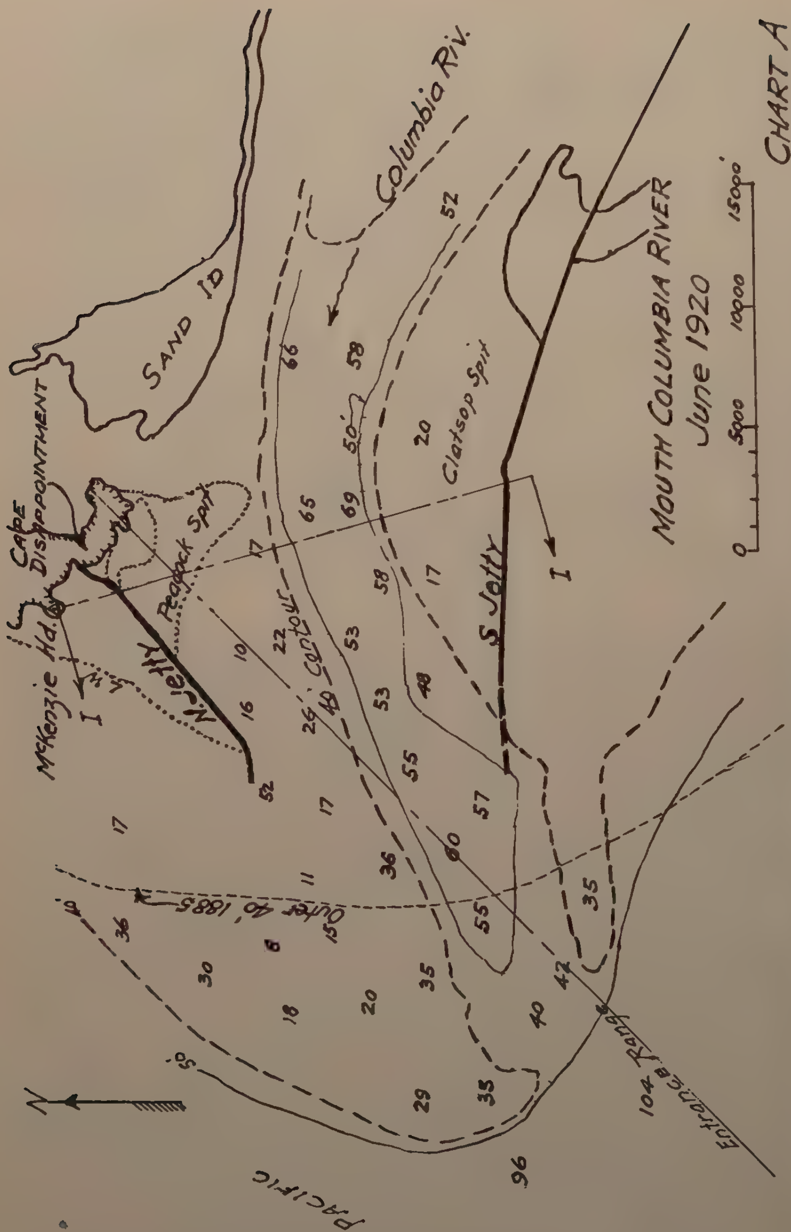


CHART A
MOUTH COLUMBIA RIVER
June 1920
0 5000 10000 15000'
Columbia Riv.
SAND ID
CAPE DISAPPOINTMENT
McKenzie Hd.
N. Jetty
Peacock Spit
Clatsop Spit
S Jetty
40' Contour
Outer 40' 1885
Entrance Range
PACIFIC

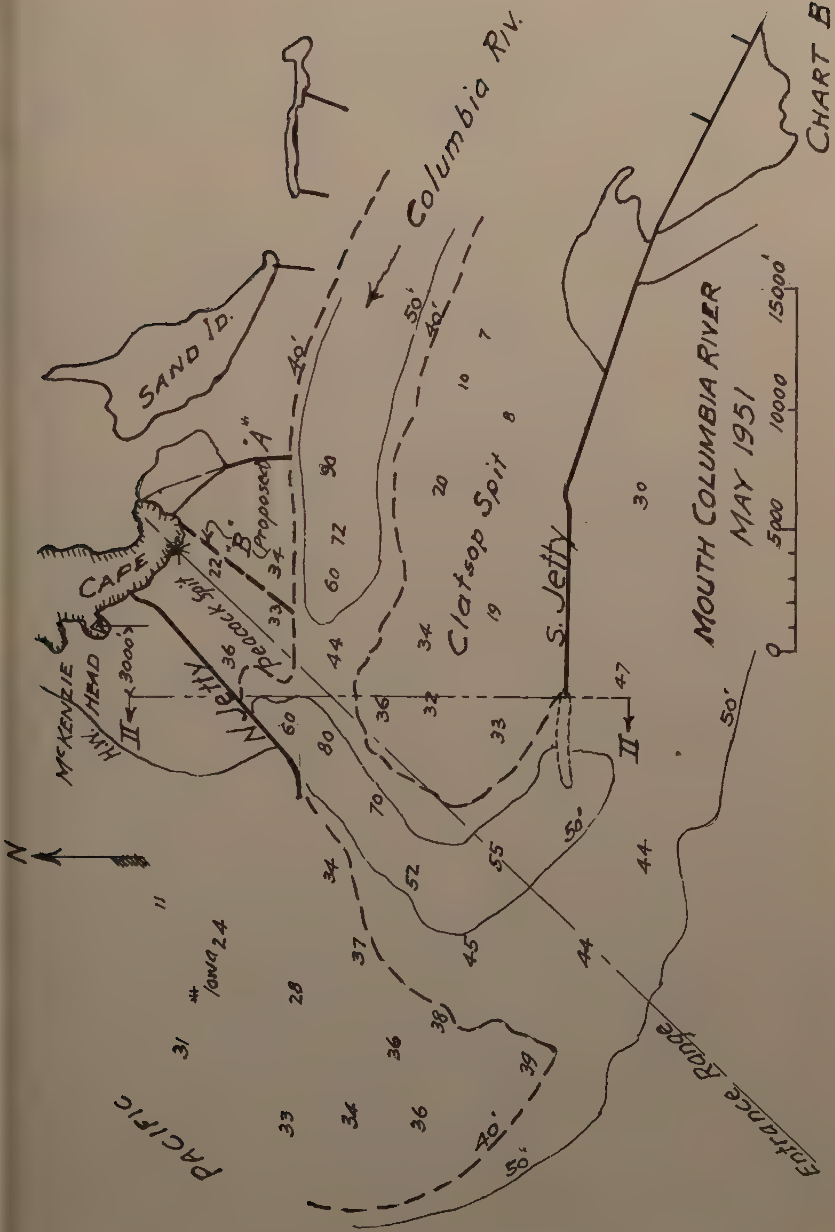


CHART B
MOUTH COLUMBIA RIVER
MAY 1951
0 5000 10000 15000'
Columbia Riv.
SAND ID.
CAPE
McKENZIE HEAD
Hill
Peacock Spit
Clatsop Spit
N. Jetty
S. Jetty
"A" (Proposed)
"B"
3000'
II
Entrance Range
PACIFIC
Iowa
N
40'
50'

(See charts in current survey, 1932.) The tip of the salt water wedge is not well defined and moves upstream with the tide probably for a distance of 10 to 15 miles. It certainly is not at the location of dredging on Clatsop Spit except perhaps for a very short time on each tide. Salinity measurements through a tide cycle show that most of the salt water is washed out by each ebb tide. (See Fig. 3) However, it appears to the writer that it matters little in this case whether the water is salt or fresh so far as maintenance of depths is concerned. The controlling factors are the strength, direction and duration of bottom velocities.

It is not agreed that littoral currents are from the north, as there is much local evidence to the contrary. It is probable, as has been held by authorities that direction of the littoral current in this vicinity changes seasonally with that of the prevailing winds. It is generally known that navigation buoys which break loose in the Columbia River are picked up on the north beach. Columbia River winter drift also travels north. There is also the case of a boy drowned in Rogue River (southern Oregon in February), whose body was found on Tillamook beach, 250 miles north after a period of 10 days. The accumulation of sand outside the north jetty is not necessarily proof of a southerly drift. There are other reasons for the deposit.

On page 37 in the second paragraph it is stated: ". . . A portion of this (littoral) material may be carried through the entrance by flood tides and waves and deposited so as not to be removed by ebb tides . . ." Any quantity of material which is brought in must be passed out to sea again, as well as the large volume of material continually being brought down from up river. Otherwise the bay would soon (geologically speaking) be filled. The ebb tide is the controlling influence, and will keep the channel open. Temporary deposits of littoral material and a shifting of material may take place, but eventually all goes to sea—if not the identical material which came in, then at least other material in comparable amount, so that inside the harbor a practical balance is maintained. Attention is here invited to the fact that since 1885 the outer 40 foot contour has advanced nearly 2 miles into the sea. (See Chart A.) This advance effected by the ebb flow from the river under control of the jetties, shows the tremendous volume of material discharged by the river. This is a continuing movement.

At the bottom of page 36 and first paragraph on page 37, reference is made to the Current Survey of 1932, and a "reanalysis" of some of the findings of that survey. The statements that the bottom velocities at low and intermediate river stages for ebb and flood are essentially in balance is in accord with the observations made during that survey. But this certainly is not the case at high river stages for any part of the time. Fig. 1 herewith, which has been prepared from curves and data on plates XI, XVII and XXV of the Current Survey of 1932, shows the relation of flood and ebb bottom (9/10 depth) velocities at low, intermediate, and high river stages. The great preponderance of ebb velocities at high river stage is very evident. The flood bottom velocity varies practically on a straight line relation, but inversely as to the river stage, as might be expected. Over 5600 individual current meter observations at 5 boat stations, were taken during the 1932 survey, at three different river stages, so the data secured can be accepted as reliable. (See Fig. 2 showing some velocity curves for high river stage.)

Considering only bottom velocities in excess of 0.6 fps does not change the conclusion that the ebb velocities are predominant. Table A, for which figures have been taken from Charts III A, B and C, shows the relative duration of

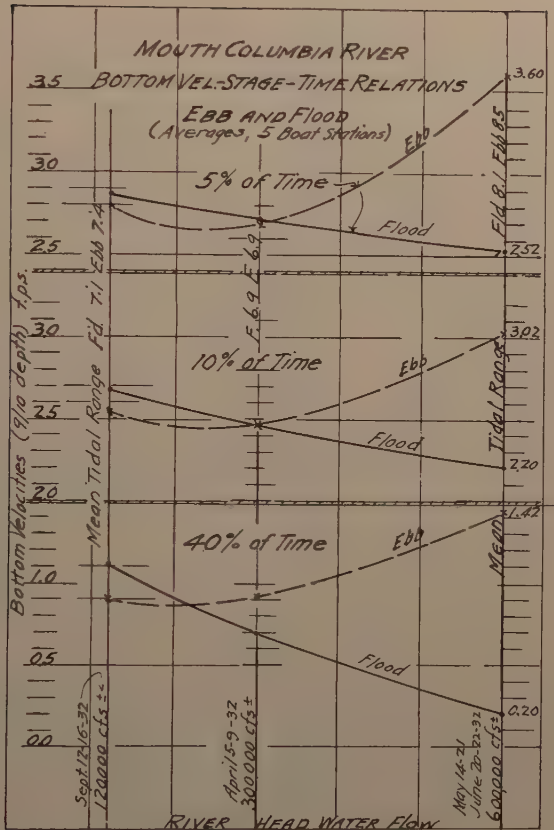


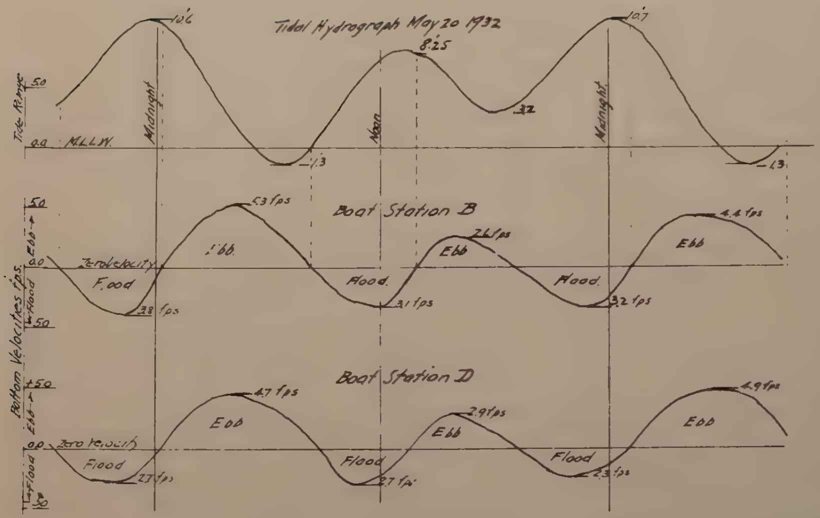
FIG. 1.

bottom flood and ebb velocities above 0.6 fps at the three river stages. Considering velocities in excess of 1.5 fps, the ebb is even more predominant.

The eroding and transporting power of flowing water (the shear stress) varies with some power of the bottom velocity. Authorities are not entirely agreed as to which power of the velocity applies, but it is probably in excess of the second. Using the square of the velocity as a conservative figure, the scouring action at high flood stages becomes very potent, so that even though the higher velocities may prevail for only a small part of the time, their overall effect (in this case) is still predominant.

Table B has been prepared to show the effect of squaring the bottom velocity, and then weighting for the duration on an annual basis to cover all river stages. In this computation and tabulation the bottom velocity V_b has arbitrarily been taken as half (1/2) that measured at 9/10 depth.

At the bottom of page 37 the author expresses doubt as to the effectiveness of the proposed jetty "B", stating that ". . . considering the jetty structure as



NOTE: Copied from Chart XIII Survey of 1932.
Curves for boat stations A-C-E are similar.
Excess of ebb over flood is evident.
Flood bottom flow does not continue through Ebb phase.
All curves are based on actual observations.

COLUMBIA RIVER
TIDE AND BOTTOM VELOCITY CURVES
BOAT STATIONS B and D
Survey of 1932.

Fig 2

a contraction work whose greatest effectiveness would occur during the high flow period, it is doubted that jetty "B" would be instrumental in reducing or affecting to any material degree heavy shoaling such as occurred during the relatively low-flow periods of the winters of 1956-57 and 57-58 . . ." This is not considered to be sound reasoning.

It is agreed the effect will be more pronounced during periods of high river flow, but this does not mean that beneficial effects would not also be realized at low river stages. $V = Q/A$, and since an increase in V and the bottom shear stress is needed a (large) reduction of the total area now available for ebb flow in the bend would result in higher values of V in the channel and over Clatsop Spit at all river stages. The critical velocity necessary for maintenance could thus be attained as is evident for the channel in the vicinity of Jetty "A" (Chart B). It should be noted also that the average rate of discharge is only about 11.3% greater during high than low river stage. (Author's Table 2.) Discharge of the tidal prism constitutes the major part of flow at any stage of river.

There are certain critical velocities necessary to prevent deposits, and the shoaling in the channel during the winters referred to by the author is due to the slightly lower velocities at the winter low stage of the river, combined with heavy wave action, etc. Increasing the velocities at low stage along the edge and on top of Clatsop Spit by control works in the bend would tend to restore the good alignment which existed from 1916 to 1932, when there was a sand bar in the location of the proposed jetty "B". Construction of jetty "A" in 1939 (see Chart B) stabilized the channel at that point, caused a widening to the south (800') and presently maintains depths in excess of 50' for a distance

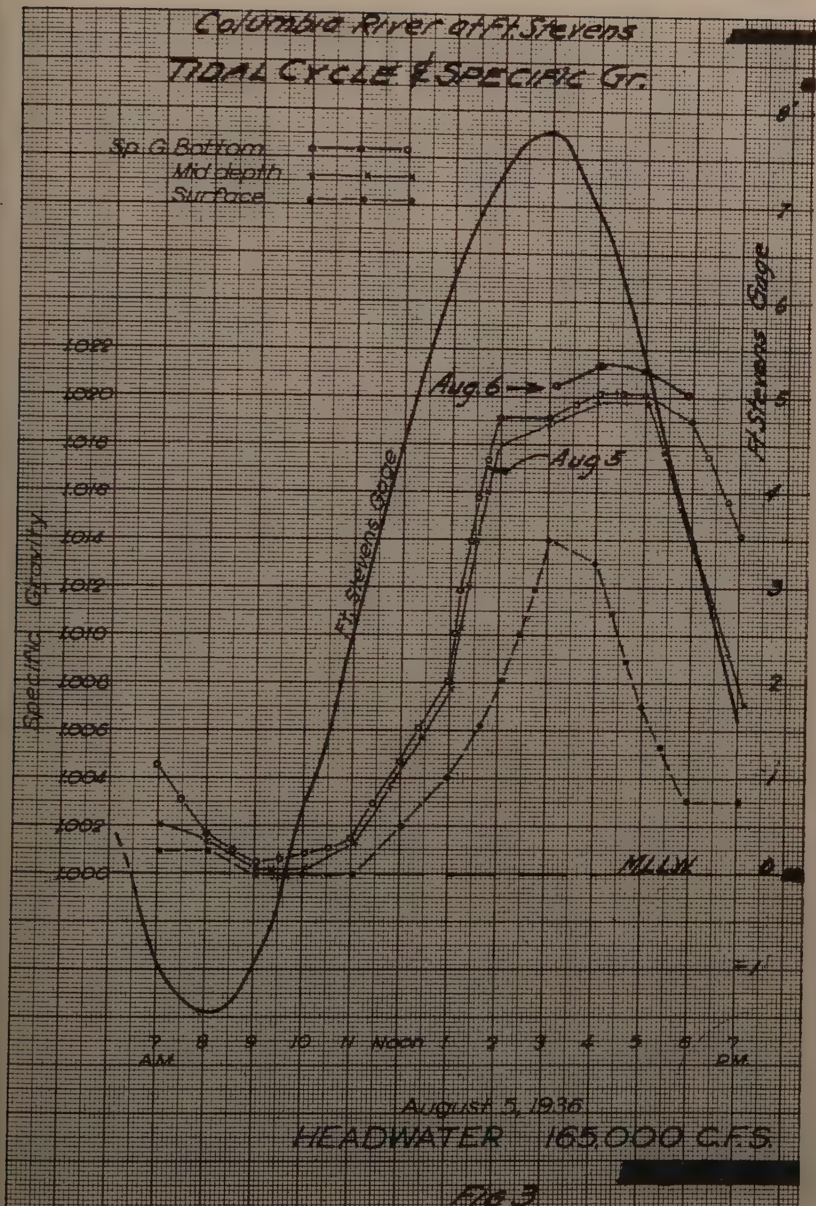

Columbia River at Ft. Stevens
TIDAL CYCLE & SPECIFIC GR.
Sp. G. Bottom
Mid depth
Surface
Aug. 6
Aug 5
Ft. Stevens Gage
Ft. Stevens Gage
Specific Gravity
1.022
1.020
1.018
1.016
1.014
1.012
1.010
1.008
1.006
1.004
1.002
1.000
MLLW
8'
7
6
5
4
3
2
1
0
-1
7 8 9 10 11 Noon 1 2 3 4 5 6 7
AM
PM
August 5, 1936
HEADWATER 165,000 C.F.S.

Fig 3

Table A

Percentages of Time Bottom Velocities (at 9/10 Depth) Exceed .6 Foot per Second
(Average of 5 stations)

River Stage	Boat Station	Time above .6 fps Flood %	Time above .6 fps Ebb %	% of Time .6 vel. favors Ebb	% of Time .6 vel. favors Flood
High	A	33.5	51.0		
	B	38.5	47.0		
	C	35.0	51.5		
	D	35.0	52.0		
	E	35.5	55.5		
	Average	35.5%	51.4%	16.9%	
Intermediate	A	37.5	43.5		
	B	42.5	42.5		
	C	42.0	45.5		
	D	38.5	43.5		
	E	43.0	45.5		
	Average	40.7%	44.0%	3.3%	
Low	A	46.5	44.5		
	B	45.5	46.5		
	C	47.5	41.5		
	D	47.5	36.5		
	E	45.0	48.0		
	Average	46.4%	43.4%		3.0%

Table A

Weighting above time values of velocities above .6 fps for approximate annual duration of each river stage:

Stage of River	Duration Months per Annum	Weighted values Ebb & Flood		Total Annual Relative Time Values Ebb	Total Annual Relative Time Values Flood
High	1½	16.9 x 1½	25.3 Ebb)		
Intermediate	6	3.3 x 6	19.8 Ebb)	45.1	
Low	4½	3.0 x 4½	13.5 Fld		13.5

Annual Duration Ratio of Ebb to Flood
for bottom velocities above .6 fps is 3.34 : 1
for velocities above 1.5 fps ratio becomes 4.54 : 1

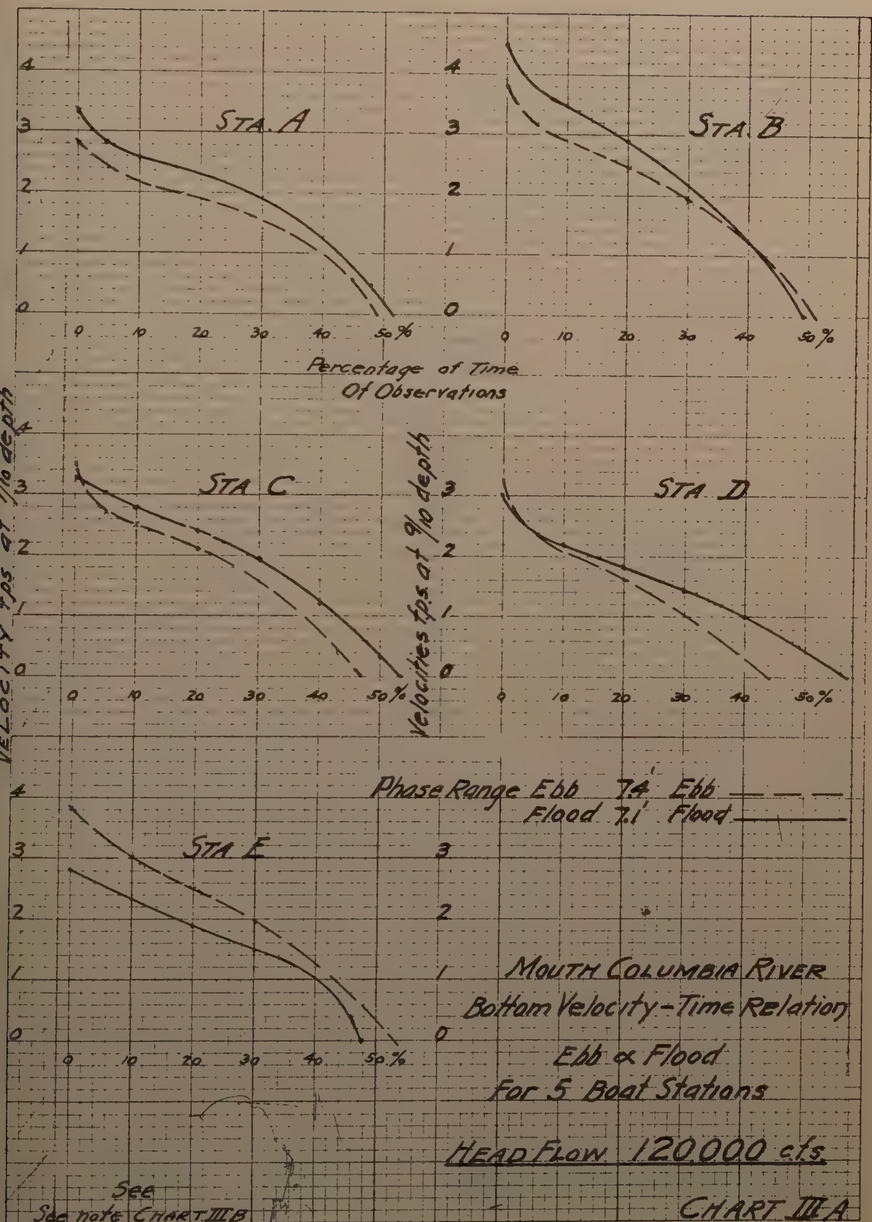
of over a mile downstream. If the ebb flow is kept out of the bend, west of jetty "A", it will scour a way for itself where it will be beneficial. This is the way a stream works, as has been proved many times and at many places.

The author's Table 1 is a good record of dredging and estimated scour and fill over a period of 2 years. This table shows a definite shoaling during the winter months when dredging cannot be done. This winter shoaling after the dredging season has been recognized for years, and presents the unsatisfacto[ry] situation that full depths, even though obtained by dredging in the summer, ar[e] not available to shipping in the winter months, when most needed.

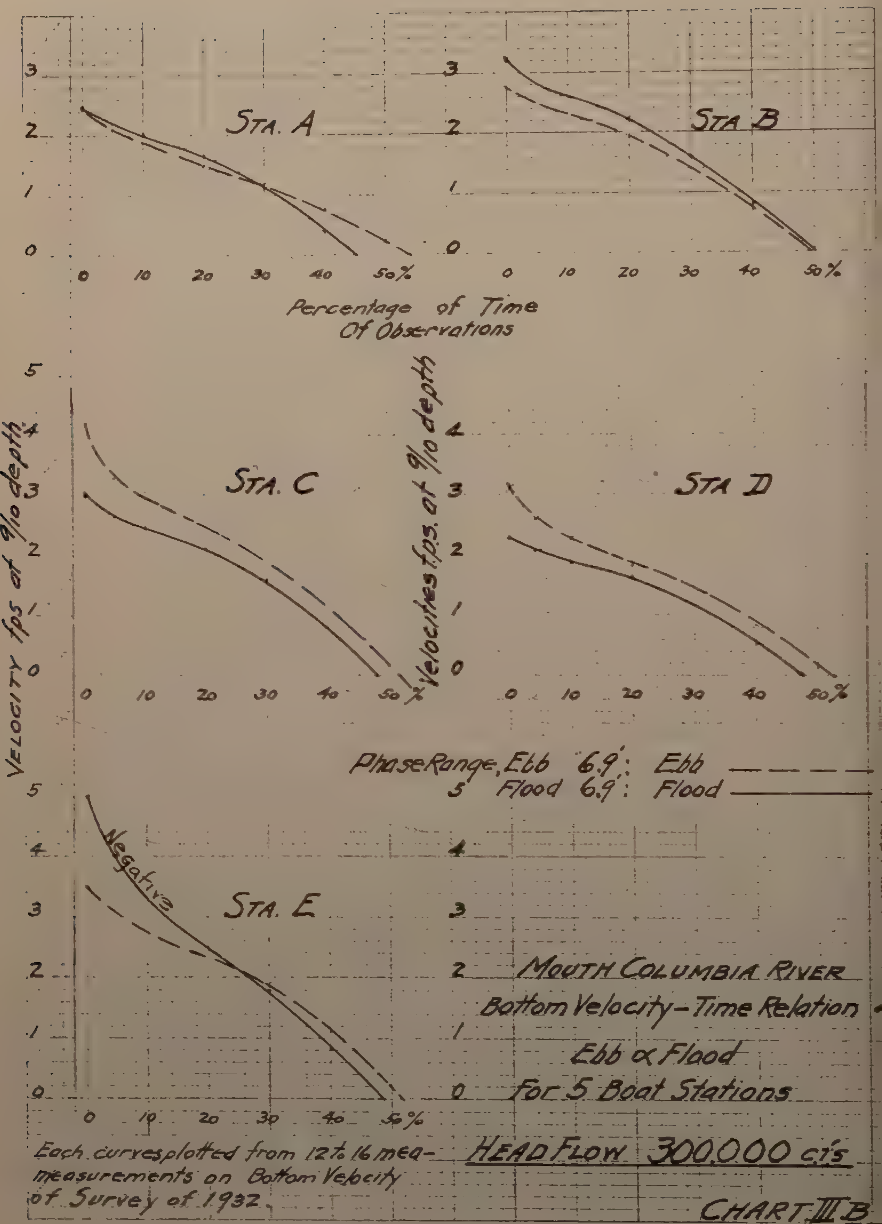
The solution very evidently is to stabilize the north bank alignment, and at the same time reduce the sectional area so that ebb velocities and bottom shear stresses will be increased to those necessary for maintenance of channel depths, even in the winter season when flows are at a minimum.

With reference to the three questions posed by the author near the top of page 38, it is thought answers can be found in the preceding discussion.

Under "Plan of Action" for obtaining additional information, it appears tha[t] the taking of ". . . observations of current direction and velocity for a continu-ous full tidal period of about 25 hours . . ." is not sufficient to cover all con-ditions. Tidal conditions and ranges change from day to day, and observation[s]



STA. A
STA. B
STA C
STA D
STA E
Percentage of Time
Of Observations
Velocity fps at 9/10 depth
Velocities fps at 9/10 depth
Phase Range Ebb 7.4' Ebb
Flood 7.1' Flood
Mouth Columbia River
Bottom Velocity-Time Relation
Ebb & Flood
For 5 Boat Stations
Head Flow 120,000 cfs.
See note Chart III B
Chart III-A


STA. A
STA B
STA. C
STA D
STA. E
Negative
Percentage of Time
Of Observations
VELOCITY fps at 9/10 depth
Velocities f.p.s. at 9/10 depth
0 10 20 30 40 50%
Phase Range, Ebb 6.9': Ebb
Flood 6.9': Flood
MOUTH COLUMBIA RIVER
Bottom Velocity-Time Relation
Ebb & Flood
For 5 Boat Stations
HEAD FLOW 300,000 c.f.s
Each curves plotted from 12 to 16 measurements on Bottom Velocity of Survey of 1932.
CHART III B

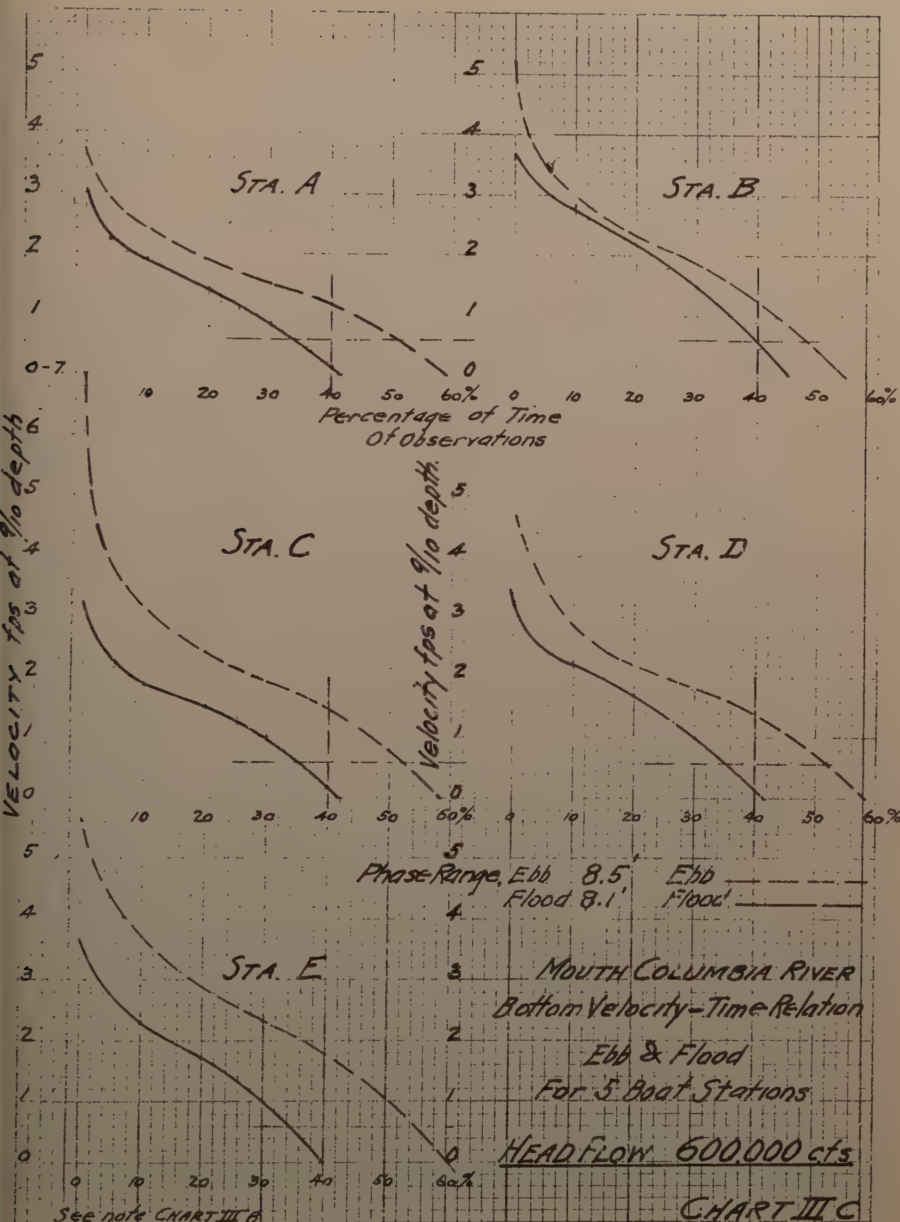

STA. A
STA. B
STA. C
STA. D
STA. E
Percentage of Time Of Observations
VELOCITY fps at 9/10 depth
Velocity fps at 9/10 depth
Phase Range, Ebb 8.5' Flood 8.1'
Ebb
Flood
MOUTH COLUMBIA RIVER
Bottom Velocity-Time Relation
Ebb & Flood
For 5 Boat Stations
HEAD FLOW 600,000 cfs
CHART III C
See note CHART III B

Table B

Comparison of Competence -- Ebb and Flood Bottom Velocities

Velocities, V_b, taken as ½ those observed at 9/10 depth;
Tractive force taken as varying with $(V_b)^2$;
Values weighted for Duration of River Stage.

40% of total time for each, Ebb and Flood*

Tide & Stage	V_b(#)	V_b^2	Weight Duration	Weight Product	Sum	Effective Ratio E:F
EBB						
Low	0.45	.202	4.5	.909)		
Intermediate	0.46	.212	6.0	1.272)	2.937	
High	0.71	.504	1.5	.756)		
						1.40
FLOOD						
Low	0.55	.302	4.5	1.358)		
Intermediate	0.35	.122	6.0	.732)	2.105	
High	0.10	.010	1.5	.015)		

*40% of time is used as this is the maximum percentage for which the flood always shows positive values. (See charts III A, B, C.)

#V_b in this column are ½ values shown by curves for 40% of time on Fig. 1

should be continued at least long enough to cover both spring and neap tide conditions, at least a week.

This discussion could be extended to greater length, as there is still much to be said. Briefly stated, it is evident that the total sectional area at Clatsop Spit shoal is too large, and that alignment of the deepest water is too far to the north where it is not available for safe ship navigation. Suitable reduction of the sectional area from the north side would increase the velocities at Clatsop Spit to or above the critical values necessary for maintenance, and at the same time rectify the alignment. Past experience and study of hydrographic surveys extending over a long period of years indicate the solution.

REFERENCES

1. Changes in the Columbia River 1903-1921, Military Engineer, Vol. XIV, No. 76, July-August 1922, pg. 211 et seq.

2. Discussion of Proc. Paper 1644 on "Stability of Coastal Inlets" (May 1958) by Brunn and Gerritsen. Proc. Paper 1785, Journal of Waterways and Harbors Div., Sept. 1958.

3. Comments of March 1958 on Committee on Tidal Hydraulics, Technical Bulletin No. 2, Reply from Committee, June 12, 1958 and Rebuttal on same, July 3, 1958. (These comments have not been published.)

APPLICATION OF SNOW HYDROLOGY TO THE COLUMBIA BASIN[a]

Discussion by David H. Miller

DAVID H. MILLER.[1]—The Columbia River is one of the largest snow-fed rivers in the world, and its hydrology has been studied by many investigators. The problems in its large and diverse basin illustrate important aspects of the growing field of snow hydrology, a field with which the writer has been intermittently associated since 1942. The writer has worked both on basic studies and on applications, and so is interested to see how the results of the snow program, a long-range comprehensive effort, are now being applied in the Columbia Basin.

He finds that while the last quarter of the paper gives a glimpse of some applications, most of it is a tour of the program itself. It touches on some highlights of a 400-page summary report, "Snow Hydrology," which was prepared by an editorial committee (F. F. Snyder, W. L. D. Bottorf, and D. M. Rockwood, chairman) of men who had had responsibility for the Engineers' part of the program. This summary report condenses some 50 or 60 research reports that appeared over the names of 30 or 35 hydrologists and meteorologists during the decade; and so the paper by Johnson and Boyer is twice removed from the originals, which it nowhere refers to. The writer is concerned with how well the paper conveys the work and results of the program, which was a large, complex and wide-range interdisciplinary research effort, and in his view an extremely successful one. He also hopes that discussion will make the program's results even more useful to other hydrologists. The discussion deals with four areas: water balance, heat balance, observations, and end-products.

The concept of water balance, an effective way to organize a college course in hydrology, is also a powerful investigative method, pin-pointing processes, such as interception and evapotranspiration, that need further study. Early in their paper, the authors present the water balance in generalized terms, and near the end review its application to forecasting runoff of the North Santiam River. The early section is familiar material, which could have been replaced by a longer discussion of the North Santiam case, which has more value.(1)

In the writer's opinion, the water-balance approach will in time be widely applied to forecasting volume of stream flow from melting snow, and should replace the index methods now used, which have changed relatively little since the days of the great innovators in snow hydrology—Church, Clyde, Boardman and others, nearly thirty years ago.

However, one modification in index methods may be noted as having at least interim usefulness. This is based on the relation of winter runoff from low-

a. Proc. Paper 1904, January, 1959, by Oliver A. Johnson and Peter B. Boyer.
1. Berkeley, Calif.

altitude basins west of the Cascades and some meteorological variables, to spring runoff from high-altitude basins to the east.[2] This work, noted by the authors without numerical material, shows the intricate interconnections among natural phenomena that lend attraction to the study of hydrology and hydrometeorology.

An unusual point in the water balance of heavily forested basins of the Cascades is worth mention. As given in "Snow Hydrology," Table 4-4, interception of precipitation at Willamette Basin Snow Laboratory, not far from the North Santiam basin, was about 18 inches per year, or 14 per cent of the precipitation. Deficiency of catch of the precipitation gages in this experimental basin is estimated[3] in two successive years as 14 and 17 per cent. The bias caused by unrepresentative location of gages is thus nearly compensated by an opposite bias caused by inadequacies of the gage itself. This is an enviable situation, though it doesn't quite bear out the author's statement (page 75) that in the North Santiam basin interception is "small compared to the total rainfall . . ."

The section on liquid water in the snow pack (pages 72-74) would carry more weight if it gave real illustrations from the Columbia Basin instead of hypothetical cases; otherwise the discussion adds little to work already presented by Gerdel[4] and others.

The heat-balance concept is discussed at length by the authors, because a principal result of the snow program was the demonstration that the findings of glaciologists can be applied, without intervention of adjustments, corrections, or "fudge" coefficients, to hydrology of snow when the physical environment of the snow is considered. The differences, for example, between physical geography of a cloudy, barren plateau in Spitsbergen and of a sunny, rugged, partly forested basin in the Cordillera can be measured and their influences on melting rate specified. Snyder[5] shows the value of the heat balance in organizing problems of snow-melt runoff, and others[6] show the precision it affords in its ability to specify hourly variation in stream flow from hourly weather observations.

For heat-balance studies, short-wave radiation was routinely observed at two of the snow laboratories, but long-wave radiation could not be observed until the Gier-Dunkle radiometer was available. Thus an early paper by the writer[7] presents a heat balance in which long-wave radiation is somewhat over-estimated, although the amount after revision is still the equivalent of 0.9 inch of snow melt in a day.

The downward current of long-wave radiation is complex, and Eq. (7) may appear over-simplified. The reason it works even approximately lies in the vertical distribution of water vapor in the atmosphere; in mountain regions during clear weather most of the vapor lies near the earth's surface in the concavities of a drainage basin. It emits a large fraction of the total downward flux of energy in the long wave-lengths, in proportion to its temperature, which is indicated by the temperature term in Eq. (7).

In the example (page 78) from the Boise River basin, the authors state that long-wave radiation is included in a term of Eq. (38), but it is not clear whether it should be computed as in Eq. (7) or in some other way.

Determining the amount of heat the snow receives by absorbing short-wave radiation requires knowing the value of the snow's albedo (a word now made familiar by the weather satellites). As the snow weathers, its albedo decreases—one of the variations that makes snow hydrology more complex perhaps than bare-ground hydrology. This decrease is obviously not a result of mere

passage of time, but of physical changes within the snow; the curves in Fig. 1 on page 70, taken from a paper by the writer,(8) should be used in combination with qualifying data, which are given with the curves in a later work.(9) A hydrologist can then estimate albedo values appropriate to the particular situation.

In Eq. (12) and others, it is not clear why the levels of ten and fifty feet above the snow surface were chosen as standard. They are not usual heights for meteorological observations; in fact, in a forested basin, fifty feet would be within the crown zone.

Between the series of equations that give melting rates for each mode of heat transfer, Eqs. (16), (17), and (18), and the following series that give melting rates in basins of various degrees of forest cover, there seems to be something missing. Except under unusual conditions, values computed by the first series for a heavily forested basin (F equal to unity) and clear skies (N zero), do not agree with those computed from Eq. (19). Data from the period 9-13 May 1949 in Willamette Snow Laboratory(10) were taken as typical of a clear-weather period: air temperature 60 degrees and dewpoint 46 degrees; the factor for wind exposure (k) not discussed by the authors, was taken as the value at bottom of page 69, 0.3. The two sets of equations are consistent if wind speed at fifty feet above the snow averages around twenty miles per hour, an uncommon clear-weather situation, especially with low wind speeds aloft. Further discussion by the authors of the steps by which Eqs. (19) to (22) were derived would be appreciated; the term 'statistical analysis' may mean almost anything.

In applying heat-budget formulas, it is convenient to divide a basin into "areas of homogeneous heat supply" as the authors call them, or landscape segments or facets, as physical geographers might call them. Each element of the physical geography of a basin operates differently on each channel of heat flow, and quantitative evaluation of each channel is necessary if the investigator is to achieve a complete view of what is happening. The heat-balance concept provides a framework for this study; in plot (or 'lysimeter') observations,(11) the processes are kept as uncomplicated as possible. Thereafter it is possible to investigate how topography and vegetation affects them. Finally, it becomes possible to take short-cuts with assurance, and to develop whatever labor-saving graphical and computing procedures are desired.

The advantage of the heat-balance concept is its flexibility. It is a framework within which the influences of topography, synoptic weather patterns, and vegetation can be analyzed; for example, the differing effects of forest on melting rates. If the reader takes Eqs. (22), (21), (20), and (19) in that order, he goes from an open basin to a forested one, and the heat-flow equations become more simple. (While these equations are only "typical" and presumably not for direct application, they can be assumed to be consistent among themselves.) First to drop out is the term for cloudiness (N), then other terms related to solar radiation: intensity (I_i); snow albedo (a); a slope factor (k'); and even a forest term (F). In densest forest, the terms for wind speed (V or v), and wind exposure (k) disappear. Open forest acts as a screen against short-wave radiation; a dense forest acts also as a mechanical insulator of the snow.

The forest canopy converts solar radiation, which in western mountains is a primary energy source, into long-wave radiation, sensible heat, and latent heat. In these forms energy is delivered to the snow at rates that are related to temperature and dewpoint of the air in the trunk space, as reservoir outflow

is indicated by stage in the outlet channel. The hydrologic significance of these thermodynamic processes in forest became plain during the course of the snow program, as investigators were led by experience with observations from the laboratory basins into new lines of scientific inquiry.[12] Long-wave radiation from forest canopy, being invisible, is not always recognized as important; this is part of a larger pattern of neglect of radiation in meteorology, which is gradually being rectified. Reports of individuals in the snow program, including those cited, demonstrate the role of long-wave radiation in the hydrologic scheme of things, and show some ways of assessing the influence of forests on the heat balance.

A caution may be noted about the term F, which first appears in Eq. (16). This is not the usual expression of canopy closure or vertical projectional area of the tree crowns, but a kind of slant shading, involving depth of crowns as well as horizontal extent, which the authors do not tell us how to calculate.

Observations requisite to applying the water-balance and heat-balance methods to basin hydrology are mentioned by the authors, who stress their recommendation by including it in the synopsis. The writer would like to add emphasis by discussing certain elements.

Extent of snow cover in a basin and its areal distribution are often not known. The authors mention extent in Eq. (30) but without comment on how the reader might derive this figure. The usual extrapolation of observations of snow on the ground made at a few climatological stations and snow courses, sometimes in unrepresentative terrain, into the high parts of a basin, is often recognized as inadequate.[13] The snow program studied snow-cover distribution in relation to terrain, ablation, and runoff[14] and around the same time, field offices of the Engineers began aerial reconnaissance of snow cover, an activity that continues.[15] These observations give hydrologists, for the first time, a definite figure on the fraction of a rugged watershed that is under snow. For historical flood studies or for design floods, the new observations can serve as basis for determining relations between extent of snow cover and accumulated runoff; however, the writer feels that relations he published in 1953 (Note 14) should be re-examined as more data from large basins become available.

As was mentioned, radiation observations figured in the snow program from its start, and soon proved their value. The Boise Basin study that the authors summarize on page 78 illustrates the role of short-wave and long-wave radiation in an open, sunny basin typical of the Rockies and the Intermountain region. They recommend that more stations be installed in mountain watersheds, a step that may require different instruments than those now in use. When, a few years ago, the Weather Bureau at considerable expense enlarged its network of short-wave radiation stations, it remained virtually confined to settled areas where the costly recording equipment could be cared for. Most of the data we now have from mountain areas were taken by the snow program in California and Montana, and the Bureau of Reclamation in Colorado. It is hoped that inexpensive integrating instruments will be developed, perhaps hooked up to radio-reporting equipment, as the radio-isotope snow gages are.

Long-wave radiation observations are, if anything, more sparse. There are few stations, mountain or lowland, at which this element is regularly observed in the United States—probably fewer than in Antarctica during the IGY. Once instrument problems are overcome, however, long-wave as well as short-wave radiation equipment should be quickly installed in every basin

yielding significant runoff from snow melting. Growth of such a network might parallel the spread of recording precipitation gages when rain hydrology needed them in the late 1930's.

The lack of radiation observations may be made up, to a degree, by taking advantage of the twice-daily maps of upper-air conditions, which extend, more or less generalized, over all the high basins. Flow patterns aloft have been shown to be associated with cloudiness and with the radiation components of the heat balance.(16) In conjunction with values of temperature and humidity aloft, these patterns may serve in reconstructions of the heat balance of snow in mountain basins.

With respect to records of stream flow, the last sentence of the fourth paragraph of page 75 speaks eloquently for itself. 'Action' agencies should realize that a damaged record affects not only a few research men, but also the operation of the reservoir after the dam is built.

The snow program was a testing ground for all kinds of instruments and elements of possible hydrologic significance. Each could be tried out on a small scale, and its relations to hydrologic processes investigated, a contribution of the program that included more than the elements of snow cover and radiation the writer has discussed in detail.

End-products of the snow program in the form of data have been published as yearly logs (as cited in Note 10). The large number of careful observations made by laboratory men in all kinds of weather and terrain resulted in a body of data characterized by unusually high station density, close coverage in time, and a good roster of elements; this body of data, after checking, was published in unusual fullness, along with auxiliary information on sites and conditions of observation. The fifteen logs have been useful in much later research, of which may be mentioned a study on precipitation measurements,(17) an analysis of snow-course measurements in relation to shading effects,(18) and parts of a study supported by the National Science Foundation on the joint processes that create a mountain climate.(19) Increasing use will no doubt continue to be made of this valuable collection of data.

End-products in the form of analyses have not been so fully presented. However, before discussing the problem of applying results of large investigational programs, we should distinguish between research reports and engineering reports of survey, say for a dam on the Zilch River. The usual survey report is important at the time it is written, but in a few years the dam is built or some other solution found, and the report becomes obsolete. A research report, on the other hand, stays valid a long time, because hydrologic problems recur, and it has worldwide appeal, because hydrologic problems are no respecters of boundaries. The Zilch River may produce floods affecting thousands of people, so that a survey report has great impact; a research report directly affects only a few men here and there, who are trying to understand why their basins behave as they do—but through them its indirect effect may be incalculable. The two kinds of reports differ, not in importance, but in permanence and in value beyond the immediate neighborhood or river valley. To get research results into use, the scientific community has developed machinery that preserves the results of investigations permanently and disseminates them outside the locality. This machinery is the open literature, so-called, exemplified by journals like the Proceedings of the ASCE. Another product of this machinery is the assurance that impartial observers will check methods and results, a matter of interest to the people paying the bill as well as to the research man himself.

The permanent record contains only a fraction of the results of the snow program. Contemporary sketches of the program are found in reports of the Committee on Snow of the American Geophysical Union,(20) but these are brief. "Snow Hydrology" summarizes many analyses, and contains some material not published in research reports; but it cannot show, in each man's own words, what observations he used, what steps he took, what assumptions he made, how he derived his results, and how he checked and compared them. In restricted space it does an excellent job, is well illustrated, a convenient reference work, and good for teaching, but it is meant to be a summary and does not try to replace individual original papers. Yet the writer estimates that of the fifty or sixty research reports of the program, only a dozen or a dozen and a half substantive papers on specific problems have entered the literature.

Here is the problem of an ad hoc investigation, that is, a group set up to do a single task, then disband. When it disbands, its members follow their research callings elsewhere, or go into other work in the sponsoring agency. Either way, they find it a burden, as the writer himself does, to write up their work in publishable form, and many results, including ideas not yet on paper, never have a chance to enter the permanent record.

How is this question handled by other groups, say expeditions? Ideally, the staff is not dispersed when it returns home, but works for several years preparing material for publication in journals of scientific societies or institutions. For example, the work of H. U. Sverdrup and other Scandinavian glaciologists, without which the field of snow hydrology as it has grown up is inconceivable, was published in Swedish and Norwegian periodicals of worldwide distribution. Years later, this work was easily available when American hydrologists needed it.

The Swiss snow-research institute presents a different case, in that it has a continuing existence (though emphasis shifts and at present is becoming stronger in hydrology). It publishes yearly logs and activity reports, but meets its professional obligation through open-journal publication by individual investigators.

These remarks about disposition of the end-products of large, publicly supported investigations are written in the hope that they may help hydrologists who will take part in even larger programs in the future. Such programs should include provision for data publication, perhaps by microcard, and for a summary report like "Snow Hydrology." But it is necessary also that individuals publish their own research in the journals, whether or not the program produces a series of research reports.

Where pertinent in this discussion, the writer has referred the interested reader to papers by the research men themselves, or at least to their reports in the snow program. Such references in scientific writing are not, as might be thought, made to recognize a contribution to the field; primarily, they are an obligation to the reader who may question a summary, find it too brief, unconvincing, at variance with experience, or who may feel that a conclusion has doubtful derivation or is insufficiently backed by data. He wants, and needs, the original definitions, methods of observation, steps of analysis, derivations, results, and checks, preferably in the words of the investigator.

In general, the men in the snow program put out good reports, which are well edited, detailed in data and analysis, and documented; but they are not easily available. It would benefit the field of snow hydrology if the individuals, wherever they may be now, would take on themselves the burden of getting

their work into circulation. This responsibility nobody else, including the authors of the paper under discussion, can take for them.

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THE HYDROLOGY OF URBAN RUNOFF[a]

Discussion by P. O. Wolf

P. O. WOLF,[1] M. ASCE.—The authors base their runoff calculations on a study of the effect of infiltration, retention and detention on a design rainstorm of three hours' duration. The variation of rainfall intensity within the period of the storm is based on the empirical rainfall formula of Eltinge and Towne which, for a five-year frequency, is given as[(1)]

$$i_{av} = \frac{90}{t_d^{\,0.9} + 11} \tag{3}$$

where i_{av} is the average rainfall intensity (inches/hour) over the period t_d (minutes).

For the same purpose the writer put forward, in 1957, a rainfall formula[(2)]

$$i_{av} = \frac{R}{\sqrt{T_1}} \cdot \frac{1}{\sqrt{T_d}} \quad \text{inches/hour} \tag{32}$$

where R represents a rainfall (inches) associated with the unit duration T_1 (hours), and T_d is the actual duration of the storm (hours) (which in the authors' case would be 3 hours).

If the origin of the scale of time T (hours) is taken at the peak intensity, the instantaneous intensities of the rising and falling limbs, corresponding respectively to the authors' Eqs. (1) and (2), and with $r = \frac{1}{3}$, are

$$i = \frac{R}{\sqrt{-12\, T_1 \cdot T}} \tag{1a}$$

and

$$i = \frac{R}{\sqrt{6\, T_1 \cdot T}} \tag{2a}$$

Eq. (3) has been plotted to logarithmic scales on Fig. 29 on which has been drawn a straight line with slope = 1/2 representing

$$i_{av} = \frac{1.85}{\sqrt{T_d}} \tag{33a}$$

or

$$i_{av} = \frac{14.3}{\sqrt{t_d}} \tag{33b}$$

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a. Proc. Paper 1984, March, 1959, by A. L. Tholin and C. J. Keifer.

1. Reader in Hydrology in the University of London, England, and Visiting Prof. of Civ. Eng., Stanford Univ., Stanford, Calif.

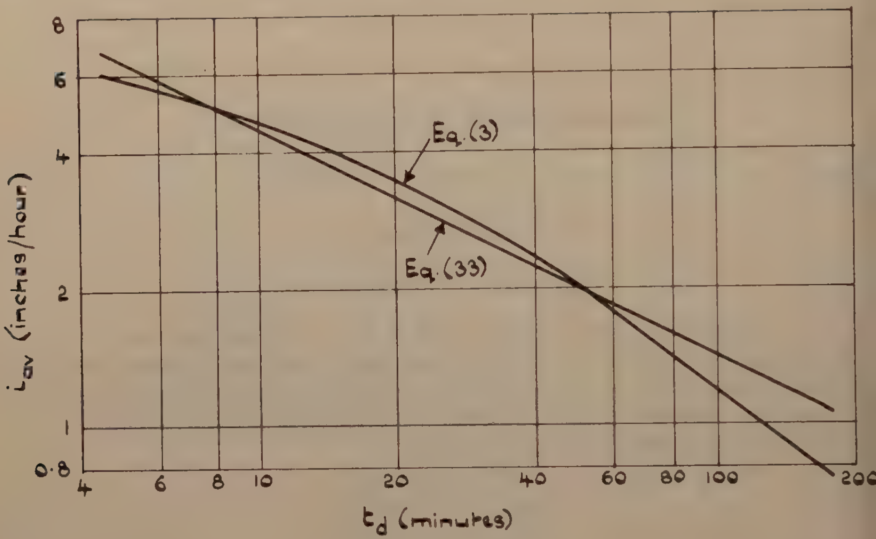

FIGURE 29

Over a range of $5 < t_d < 180$ minutes, Eqs. (3) and (33) are seen to be in fair agreement. This means that no great numerical change would result in the authors' conclusions on the flood runoff in the sewers of Chicago, from the substitution of Eq. (33) for Eq. (3), but the writer suggests that Eq. (33) is preferable because

(a) It is dimensionally consistent and
(b) It is physically more likely to be correct than (3).

The justification for statement (b) may be found in considering the values of total rainfall over periods in excess of the 180 minutes which represent the authors' upper limit. As t_d becomes large, the value of c becomes negligible compared with $t_d^{0.9}$ and the total precipitation (inches) tends to

$$P = i_{av} \cdot t_d / 60 \longrightarrow 1{\cdot}5\, t_d^{0.1}$$

which clearly represents too small a rate of increase of total precipitation with duration of storms of a given frequency.

According to Eqs. (32) and (33) the intensity i_{av} or the instantaneous intensity i would increase to infinity as T_d or T approach zero. As the variations of rainfall within the peak have no practical effect on runoff if a short enough interval is chosen, the writer's 1957 paper suggests that the time interval considered should be greater than 45 seconds. With the five-year frequency adopted by the authors, the peak average intensity would according become

$$i_{av,\ max} = \frac{14.3}{\sqrt{0{\cdot}75}} = 16{\cdot}5 \text{ inches/hour} \qquad (33c$$

for a duration of 45 seconds. Although twice the authors' maximum rate, this peak average is quickly reduced with increasing duration. In fact, the instantaneous value i = 8 inches/hour would occur at the beginning and end of a high-intensity rainfall burst lasting only a few seconds more than the suggested peak period of 45 seconds.

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TWO METHODS TO COMPUTE WATER SURFACE PROFILES[a]

Discussion by T. R. Anand

T. R. ANAND,[1] A. M. ASCE.—The paper presented by Messrs. Lara and Schroeder was studied with interest. The necessity of backwater information for the various purposes outlined by the authors is indeed felt in hydro-electric development. To their list may be added: Design of bypass and upstream cofferdams, determination of compensation to upstream plants already existing if a new development is likely to drown out their tailraces, determination of gain in head by tailrace excavation, etc. etc.

The precise determination of eddy losses is difficult and for this reason our practice has been to include it in determining velocity head differences. According to our practice, if velocity increases going downstream, the full value of the difference of velocity heads $\frac{V_2^2}{2g} - \frac{V_1^2}{2g}$ is added in working out upstream levels. In other words, no eddy loss is considered for this condition. If velocity decreased downstream, the recovery of head is assumed 50% of the value $\frac{V_2^2}{2g} - \frac{V_1^2}{2g}$ and is subtracted to get upstream water levels. Thus, eddy loss is considered to be 50% of the difference of velocity head recovery.

Fig. 1 shows a comparison of some values calculated according to these assumptions with actual values measured two years later. This river reach is 3600 feet long and has eddy losses due to small rapids in the lower portion and an island in the middle. The maximum difference between computed and actual values is only 1.0 foot, where a big island divides the flow. In other reaches the difference is only 0.5 foot or less.

As regards bend losses, the following formula is quite good:

$$\text{Bend loss} = \text{coeff} \times \frac{1}{2}\left(\frac{v_1^2}{2g} + \frac{v_2^2}{2g}\right)$$

the coefficient depending on the bend angle.

When a bend includes more than one reach, the bend loss coefficient for the entire reach is determined and distributed to each reach as a proportion of the bend angle in that reach. However, if a value of 'n' is carefully determined from gauge readings and other information, it generally includes the effect of bend losses. Where the bends are sharp and backwater information is required within, or just upstream of the bend, separate computations may be justified.

The theory behind the step method for backwater curves is well established and the use of Manning formula is popular too. The concept of carrying

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a. Proc. Paper 1997, April, 1959, by Joe M. Lara and K. B. Schroeder.

1. Engr., The Shawinigan Eng. Co. Ltd., Montreal, Que., Canada.

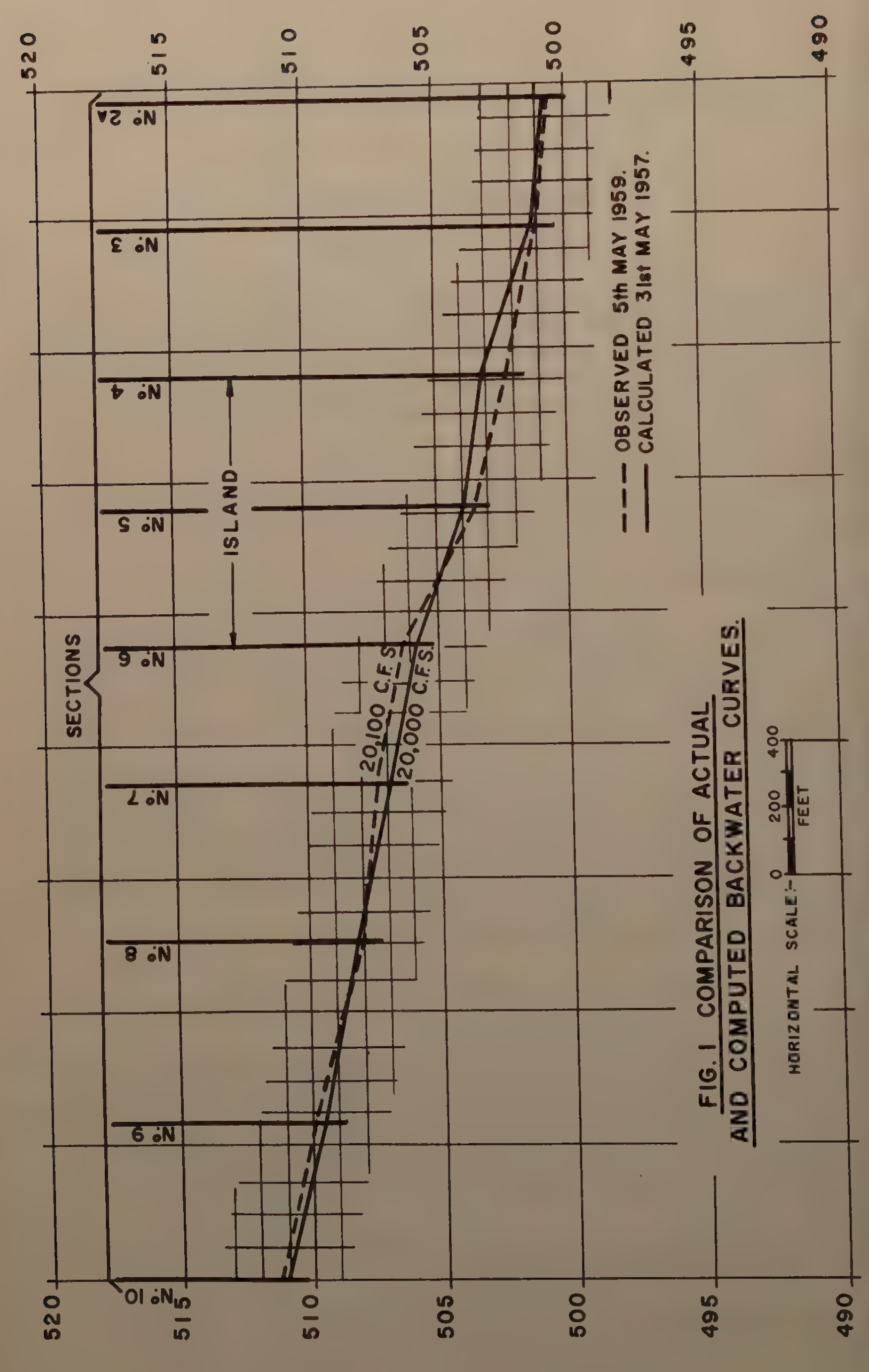

FIG. 1 COMPARISON OF ACTUAL AND COMPUTED BACKWATER CURVES.

capacity K was developed, I believe, by Boris A. Bakhmeteff[1] and is basic to his backwater theory and methods outlined by him. The incorporation of this concept into the step method would be useful and time saving in case of rivers consisting of a deep section and one or two overbank portions, as suggested by the authors. The ordinary standard step method becomes a bit involved in such a case, inasmuch as separate values of discharges have to be assumed for each distinct channel and adjusted until an agreed upstream level is obtained, as suggested by Woodward and Posey.[2]

Sometimes several values of 'n' have to be tried in different reaches of a river in the course of one backwater study. In such a case, use of plots $A\,R^{2/3}$ vs water depth is more useful than the use of K curves.

$$K = \frac{1.486}{n} A\,R^{2/3}$$

It can be seen from the above equation that if K curves were used in such a case, a set of curves would need to be plotted for each value of 'n'; if $A\,R^{2/3}$ curves could serve various trials. A good example of this case occurs where 'n' for a certain reach is determined from known value of discharge and gauge readings at a few stations within the reach by backwater calculations.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division nsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air ansport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hy- ulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil chanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors W), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the sym- s (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning h Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To ate papers in the Journals, the symbols after the paper number are followed by a numeral designating issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 59 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics ision during 1958.

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Y: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c]. 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

E: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

Y: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

GUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c], 2140(SM4)[c].

[c] Discussion of several papers, grouped by divisions.